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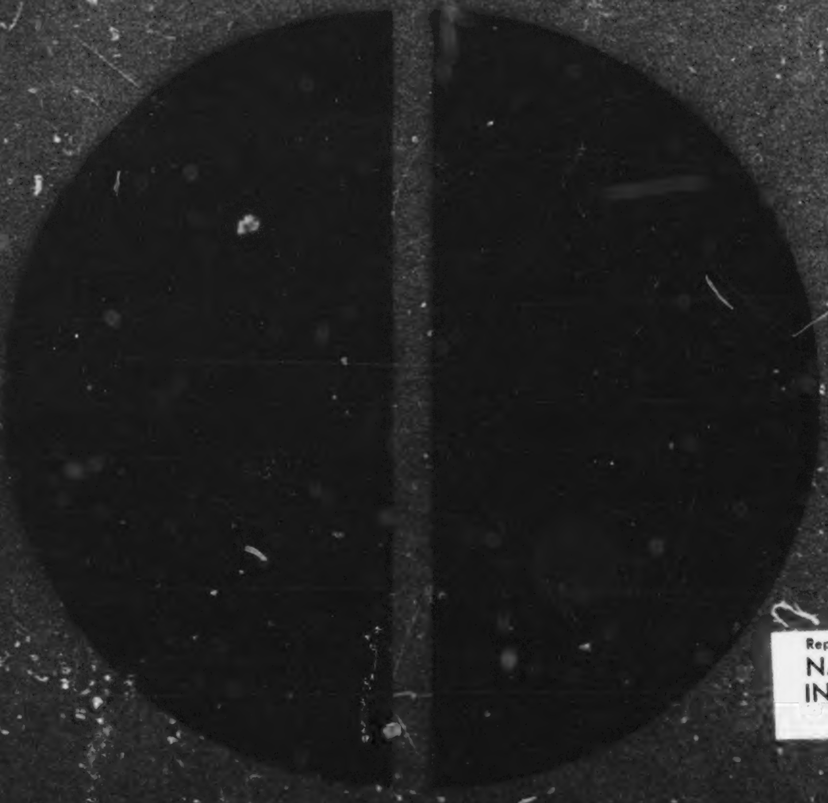
2nd
international symposium for waste treatment lagoons



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WELCOME TO THE SYMPOSIUM

Arthur F. Williamson
Director, Sanitary Engineering Services
Wyoming Division of Health and Medical Services

It is my pleasure to welcome all registrants, participants, helpers, students and others present to this Second International Symposium for Waste Treatment Lagoons. Normally we expect the Mayor of the city to make the welcoming address but in this particular case the responsibility has fallen to the Chairman of the Symposium Committee.

I am sure that the next few days will be productive to all people in attendance and I am also sure that the proceedings and the State of the Arts Paper which will be produced from this meeting will be of great technical value through the years to come.

As the welcoming speaker and chairman of the committee I feel it is my prerogative to make a few general comments relating to the Symposium. As you will note from the program, this Symposium is sponsored by the Missouri Basin Engineering Health Council, in conjunction with the Federal Water Quality Administration. Many people may not be familiar with the Missouri Basin Engineering Health Council. This Council has been in operation for quite a number of years and consists of the State Sanitary Engineers and the Water Pollution Control Administrators of the ten States included in the Missouri River Drainage Basin. A representative from the Federal Water Quality Administration and from Health, Education and Welfare are also participating members. The basic purpose of the Council has been to provide a working organization whereby the States could meet once a year and review and discuss problems that were common throughout the Missouri River Basin.

The Council has been deeply interested in waste stabilization lagoons as we consider the Missouri River Basin as the birthplace of such waste treatment facilities. This method of treatment originated in the upper portion of the Basin in the late 1940s and has progressed steadily since that time.

The First Symposium on Waste Stabilization Lagoons was held here in Kansas City in August of 1960. The First Symposium was designed to provide an evaluation of the waste stabilization lagoon as a method of waste treatment and, also, to develop acceptable design criteria for such installations. The First Symposium was a successful meeting and its value is still reflected, as the proceedings of that meeting are still utilized as primary reference material. I might also say that the proceedings of that meeting have been through two printings and are still rather difficult to obtain.

In the later years, following the First Symposium, the Missouri Basin Engineering Health Council has realized there would be need for updating our information on waste stabilization lagoons sometime in the near future. We also realized there were many innovations appearing in the waste treatment lagoon field and there was definite need for information on these new installations. We have seen aerated lagoons come into general use. In many areas anaerobic lagoons have appeared to be the reasonable solution; and, of course, any time waste sludge is to be handled, first considerations are usually given to some method of lagooning. In all of these areas there is a very definite need for the evaluation of results and for the development of firm design criteria if we are to continue to progress in these fields.

As a bit of sideline information for those individuals who have not read the latest issue of The Economics of Clean Water, Volume I, published by the Federal Water Quality Administration, I would point out that their summary shows that 27.9 percent of all municipal waste treatment facilities are classified as lagoons. With a total of 12,374 installations listed, this would mean there are approximately 3,452 lagoon installations serving municipalities.

Approximately three or four years ago the Missouri Basin Engineering Health Council felt there was a very definite need for a second symposium on the matter of waste stabilization lagoons. The usual committee was appointed and work started on finding the necessary financing and development of the program. In the summer of 1969 a research grant was obtained from the Water Pollution Control Administration for the purpose of holding this Symposium on Waste Stabilization Lagoons and also to provide a State of the Arts Paper on the subject. With the matter of financing arranged we then moved into the field of amassing the necessary information. For this purpose we developed a work committee which represented all areas of interests and consisted of representatives from federal agencies, state governments, municipalities, consulting engineers, industry and the universities. All known areas were searched for information and it became the job of our Project Director, Dr. Ross McKinney, to sort out this material and develop the program. I am certain there is still much good information in people's files that we did not receive, that could add much to a conference such as this. However, the results were gratifying. I am sure you can see from the agenda that it was necessary to run dual programs in an attempt to cover the material which we thought would be of greatest value.

In our over-all planning we felt that there were several goals that we should strive for.

1. In the field of waste stabilization lagoons we felt there was a definite need for re-evaluation of existing design criteria, operating methods, etc. Generally there has been universal acceptance of this method of treatment; however, we felt current evaluation is necessary. I would like to point out that the committee made special effort to obtain pro and con information. If there are problems with the use of waste stabilization ponds, these problems should be known so that

proper consideration can be given.

2. In the field of aerated lagoons, we felt there was a serious need for definite and firm design criteria. Within the past few years practically all States have been faced with the installation of aerated lagoons. Without some information on acceptable design and operation, and expected results, it makes the problem of approval somewhat difficult. There is a very definite need for evaluation of results, operating methods and design criteria for aerated lagoons.
3. Within certain industries anaerobic lagoons have provided a very reasonable method of treatment. Again we are dealing in an area in which firm design criteria and evaluation of results are lacking.
4. While we are looking at the whole field of lagoons, we felt it would be well to consider sludge lagoons and amass whatever information was available in this area. Such units are probably not widespread; however, they are frequently encountered in industrial and municipal operations.

It is hoped that this Symposium and the State of the Arts Paper will provide the information that we feel is lacking in the field. The Symposium has been designed to gather the most possible information. We have allowed adequate time for discussion of papers, and we have provided special rap sessions in the evening where all items can be discussed. Only through these informal discussions can we seek out the mass of information that is available within this group. We hope that everyone wholeheartedly participates in these discussions.

As a closing comment I would like to point out that I was unable to attend the First Symposium on Waste Stabilization Lagoons that was held in 1960. At that particular time I was selling waste stabilization lagoons as a treatment method to several countries in South Africa. I might say that it was a successful selling trip and that lagoons are quite widely utilized in that area at this time. There was one unexpected side-benefit from that trip. During the tour I was closely associated with a young engineer, and I might say soils engineer, who had been assigned to work with us to carry out necessary laboratory work on existing lagoons and to generally help out in the program. This young engineer continued on in the field of sanitary engineering and will present two papers during the Symposium. I particulary want to welcome Ghert Marais from Capetown, South Africa to this Symposium.

WASTEWATER TREATMENT AND WATER POLLUTION IN THE UNITED STATES

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For
Water Quality and Research
Washington, D.C.

I believe that the need for a new approach to the treatment of sewage water and the disposal of other wastes in our no deposit-no return society is at last beginning to penetrate the consciousness of the American people.

Indications of this can be found on all sides!

President Nixon called attention to the tremendous problems in this area last New Year's Day when he warned that "the 1970's absolutely must be the years when America pays its debt to the past by reclaiming the purity of its air, its waters, and our living environment. It is literally now or never."

Young people especially look upon environmental problems as a particularly unwelcome legacy to be inheriting at this time. Indeed, there should be no generation gap in our fight for a cleaner environment. It is to everyone's advantage to work for cleaner air, unpolluted water and less accumulation of solid wastes.

I am sure that you have voiced at least some of these sentiments during recent months.

Although waste disposal problems have been with us ever since the first settlers arrived on our shores some 350 years ago, they did not become a potential threat to our very survival until relatively recently. It is largely the tremendous increase that has occurred in our population since World War II that makes this issue so critical and insistent of solution.

Let's review a few facts to put this point in proper perspective. Remember that back in 1940—That's just 30 years ago— There were fewer than 135 million Americans. Even then we had significant waste disposal problems. Today—we are in the process of taking a new census which may show our population to be well above the 204 million officially counted. Now that's a 50 percent increase in population in 30 years. In the next 30 years—by the year 2000—that figure is expected to raise by another 50 percent to some 300 million.

Not only are the numbers of people rising at an explosive rate, but so are per capita consumption of water and other resources ancillary to waste production. All the while, our known sources of water remain essentially the same. In other words, we must continue to draw upon the same water resources for a population of 204 million and more than we had when our Nation numbered 135 millions and less.

What this all amounts to is that we must live with the conditions we create. But, those conditions are becoming almost unmanageable because of our growing numbers. Let's face it—the raw municipal sewage that could be dumped almost with impunity into a river or lake in 1940 because its volume was not great enough to cause serious problems today could well be choking that same waterway to death even after primary treatment.

The only answer to this mounting problem is increasingly more efficient systems of disposal for all our wastes, leading, I believe, to eventual recycling and reuse.

The challenge of this problem is for us in the field of water pollution control to strengthen our efforts at all levels of government—Federal, State, and local—to do a better job. This can come about by better cooperative effort and is certainly dependent upon good communication between us. We must strive to work as a single team

We in the Federal establishment must and will continue to press for higher water quality standards among the states and for the strict enforcement of those standards which have already been fixed. We will continue to seek funds for the construction of municipal pollution control facilities and support research aimed at abatement of pollution.

One area of concentration in research is on methods to control the more complex problems of combined storm and sanitary sewers.

Control of these discharges has been generally neglected until recent years because the only method of solving the problem was believed to be the separation of combined sewers into storm and sanitary sewers—A costly and disruptive process. Through the efforts of the Federal Water Quality Administration's research program, however, a new concept for the control of these discharges is being developed.

One of the plans being tested is the storage of excessive flows until they can be released to treatment plants. Fullscale storage facilities are being built in Boston, Milwaukee, Dallas and Shelbyville, Illinois. The evaluation of their effectiveness and costs will allow other communities to design similar combined sewer-pollution abatement facilities more economically.

In the treatment phase, also, there are alternatives. Screening and an air flotation system are two treatment methods that were devised to deal with widely fluctuating high flow rates and the dynamic quality changes of combined sewer overflows. Studies on these continue.

A novel rotating collar, vibratory base screening treatment unit for combined sewer overflows was demonstrated last year in Portland, Oregon. The unit provided primary treatment to normally bypassed sewage at a cost only slightly higher than the equivalent conventional treatment. This is because the space utilization of the screens is only one-tenth that of settling tanks.

Through studies such as these, a combination of control methods is being developed which we hope will be applicable to the various combined and separate stormwater-sewer problems being experienced throughout the country. Although estimates of the cost of controlling discharges by these new methods are very preliminary, they may turn out to be only about one-third of the earlier estimates based on separation.

An unfortunate by-product of rapid new construction in our urban and suburban areas is erosion and sediment accumulation in nearby rivers and streams. The National Association of Counties Research Foundation, in conjunction with the FWQA, has developed a "Community Action Guide for Erosion and Sediment Control" that is designed to help local officials develop ordinances to control pollution from construction projects. These programs involve the use of present technology, such as vegetation control, mulching, sediment traps and other common erosion control practices to substantially reduce the silt load to urban waters. Nothing really new here except the determination in some areas to take needed action.

Of special interest today is FWQA's research into the joint treatment of municipal and industrial wastes. The benefits of such joint treatment are considerable. Industries, while paying their share of operating and amortization costs would be spared the burden of full capital costs, while the regionalization of waste treatment and economies of scale can help some communities achieve much more effective pollution control.

The main point of the environmental program the President launched in February, and I believe a much misunderstood point, was his call for the expenditure of $10 billion over a five-year period for the construction of municipal waste treatment plants and interceptor sewers needed to meet our national water quality standards.

The Federal government's $4 billion share of the cost is to be allocated at the rate of $1 billion a year over the next four years, with a re-assessment to be made in 1973 for 1975 and subsequent years.

To make financing the municipal share of the program easier for local governments, the President proposed the establishment of an environmental financing authority (EFA). The EFA would buy any municipal bonds a community might not be able to sell for treatment plant construction. This is meant to make the construction of such facilities dependent, not on a community's ability to float a bond issue but on its waste disposal needs.

Other provisions in the President's environmental action program for water pollution control include:

Revision of the statutory formula governing allocation of grants for treatment plant construction to permit the construction of plants where they are most needed and where the greatest improvements in water quality will result.

Requirement that treatment plants be built to prescribed design, operation and maintenance standards, and that they be operated only by certified operators.

Requirement that municipalities impose users fees on industrial users that are sufficient to meet the costs of treating industrial wastes.

Requirement that comprehensive river basin plans be drawn up to assure that the construction of municipal treatment plants is complemented by the abatement of all other sources of water pollution.

Encouragement of the construction of integrated regional treatment facilities to effect economies and maximum abatement.

Extension of Federal-State water quality standards to include precise local effluent standards for all industrial and municipal sources.

Provision that the violation of established water quality standards is sufficient cause for court action.

Revision of Federal enforcement procedures to permit swifter court action against those in violation of water quality standards.

Provision that violation of established water quality standards is subject to court imposed fines of up to $10,000 per day.

Authorization for the Secretary of the Interior to seek immediate injunctions where severe water pollution threatens imminent danger to health or irreversible damage to the water environment.

Extension of Federal pollution control authority to include all navigable waters, both inter- and intra-state, all interstate ground waters, the United States portion of boundary waters, and waters of the Contiguous Zone.

Tripling of Federal operating grants to state pollution agencies from $10 million now to $30 million in 1975.

The Rules and Regulations recently published in the *Federal Register* requiring that comprehensive basin plans be devised before Federal Funds are committed to a pollution abatement project is aimed at tightening up each area's pollution abatement procedures. As the President said, "A river cannot be polluted on its left bank and clean on its right. In a given waterway, abating some of the pollution is often little better than doing nothing at all, and money spent on such partial efforts is largely wasted."

Thus, the new rules require that in order to quality for Federal assistance, any new treatment plant must fit in with the basin-wide program that would include data on the entire river basin's sources of pollution, the volume of discharge from each source, the character of the effluent, and the effects of existing treatment facilities in the area.

No grant would be made unless the Commissioner of the Federal Water Quality Administration determines that the proposed project is included in an effective metropolitan or regional plan certified by the Governor of the State as being the official plan for the area.

Nor are grants to be provided for any system that is designed to treat industrial wastes only. Any industry that feeds its wastes into such a system must pretreat them in order to make sure that they will not interfere with the efficient operation of the community system.

The new regulations also stipulate that wherever industrial wastes are to be treated in a new plant built with Federal aid, a system of "cost recovery" would be required, whereby every industry involved would be assessed a share of operating and amortization costs proportionate to its contribution of industrial wastes to the total cost of waste treatment.

Finally, every approved project must contain a plan for inspection and evaluation by the appropriate state water pollution control agency at least once each year for three years. Inspections after that would be as agreed to by the State and FWQA.

In conjunction with the Federal funding of treatment facilities, the states have been required to set definite pollution abatement schedules for municipalities. Many have done this. Others have not. I would urge each state to bring their schedules up to date at the earliest possible time if changes are needed because of a lack of matching funds or for other reasonable cause.

Since the initiation of the Federal construction grant program in 1957, it is significant to note that the population served by some degree of waste treatment has increased by more than 51 million persons. Today, more than 92 percent of the population served by sewers are connected to a waste treatment plant, compared with only 57 percent in 1956.

Great effort must be made by industry and agriculture, however, to make the pollution abatement campaign more uniformly effective. Farmers, for instance, must pay more attention to controlling the massive amounts of animal wastes that result from feeding operations.

To highlight this problem, it has been estimated that the 4.5 million or so cattle in Minnesota produce about 40 million tons of solid wastes in a year and about 15 million tons of liquid wastes. This mass of animal excretion, in turn, contains about 290,000 tons of nitrogen and some 40,000 tons of phosphorous.

Ways must be found to keep excessive amounts of sediment out of our streams, and we must devote more research to solving the difficult pollution problems caused by agricultural chemicals and fertilizers and by saline irrigation return flows.

Industrial wastes also deserve our attention, and much more must be done by industry itself to control this aspect of our technological advancement. Wastewater discharges from industries involved in the production of food, textiles, paper and chemical products, petroleum and coal, rubber, primary metals and machinery and transportation equipment together exceed 13 trillion gallons annually. By comparison, the wastewater volume of domestic sewage from our sewered population amounts to slightly more than 5 [illegible]illion gallons.

As I suggested earlier, the complementary characteristics of sewage and industrial wastes can, if properly controlled, permit more effective waste reduction within the waste treatment plant. Joint treatment also provides more effective pollution control by encouraging regionalization of the waste treatment system. And, it offers significant advantages to both municipalities and industries in terms of economics.

Industry nevertheless will be expected to reimburse municipalities for the added cost which the treatment of its wastes imposes. But, since industry will be spared the more burdensome initial investment, it should be no hardship to pay a fair share of the operating costs. At the same time, industry's contribution assures municipalities of revenues to provide for adequate operation and maintenance of their treatment facilities, and this, in turn, will help safeguard the Federal and State investments.

There is still a further benefit to the equitable cost recovery plan. Charges relating to volume and strength of the wastes discharged into the municipal system should be an incentive to industry to cut down on those wastes. This aims at one of the ultimate solutions to many of our pollution problems, which is to make fuller use of our resources rather than to discharge them and to allow them to cause pollution problems.

What we would like to prove—with the help of industry—is that we can continue to develop our technology while at the same time ensuring that it will serve our environmental needs more effectively.

As the President's task force on science policy said recently, "Our national progress becomes ever more critically dependent upon the excellence of our science and technology." Dr. Lee DuBridge, the President's science adviser, put it another way. He said, "Man isn't going to solve his environmental problems by returning to cave-man living standards, but by using his intelligence to correct his abuses."

These, then, are the goals we strive for as we work to adjust our progress to the needs of the 1970s.

THE ROLE OF PONDS IN WASTEWATER TREATMENT

M.O. Allum, Public Health Scientist, and
C.E. Carl, Director, Division of Sanitary Engineering
South Dakota Department of Health

You will note that I have altered the title of this presentation somewhat in that I have substituted the term "pond" for "lagoon". This is because our usual practice in South Dakota is to refer to these treatment devices as "stabilization ponds" or "wastewater ponds". We usually reserve the term "lagoons" for waste-holding ponds which do not necessarily provide treatment except, perhaps, settling of solids.

In this regard, it is interesting to note that even though a plea was made for uniform terminology at the 1st Symposium 10 years ago by D.F. Smallhorst of Texas(1), we have not accomplished this so far. In the program for this Symposium, the following terms are applied to man-made ponds designed for the treatment of wastes: lagoon, sewage lagoon, waste-treatment lagoon, stabilization lagoon, waste-stabilization lagoon, waste pond, oxidation pond, stabilization pond, and waste stabilization pond.

The prefixes "anaerobic", "aerobic", "facultative", "aerated", "sludge", "manure", etc., are applied as well. Perhaps we need a wide-spread opinion poll as was recently conducted to determine the exact meaning of the term "tertiary treatment". After which, we may be even more confused.

Now, one great advantage to an early spot on the program is the opportunity to nibble at the frosting of everyone else's cake! For example, Willis Van Heuvelen of North Dakota, who follows me on the program, will present information based in part on data we supplied him in answer to a very thorough questionnaire. It is tempting to include at least part of the South Dakota data in my own presentation for the combined purposes of illustrating specific pond applications and of passing some portion of this half hour with less sweat and pain. In truth, it is nearly impossible in this presentation not to preempt the information to be presented by some of those who follow on the program. If I do this, I hereby apologize, the purloinment is strictly intentional.

In the first Symposium, ten years ago, one of us presented a paper on waste stabilization ponds in South Dakota(2). The presentation ran to 17 single-spaced typewritten pages, including tables. This presentation will be somewhat shorter for two reasons, first, another ten years of experience with ponds has tempered our evaluation of ponds to a degree, and second, the purpose of this half hour is to present a broad overview of the contribution of ponds to the solution of wastewater treatment problems (which, as I interpret it, means I can omit, or ignore, the details).

In the "X" number of years since the first ponds for the treatment of wastes were put in operation in Texas or California or Montana or North Dakota or wherever (we suspect the Chinese invented ponds 1000 years ago), the application of the pond concept to a wide variety of waste-treatment problems has occurred. Perhaps two of the newer applications of particular interest in this symposium are ponds in which oxygen is in part, or in the main, supplied by mechanical aerators or by diffused air systems and ponds designed to provide treatment in the absence of oxygen. The content of the program is evident of the interest that has developed in these two applications, roughly one half of the papers to be presented in this Symposium deal with aerated and anaerobic ponds.

When one considers the role of ponds in wastewater treatment, perhaps the most striking contribution is found in the opportunity afforded to small communities to share in the comfort and satisfaction of a water-carried sewage disposal system which will provide generally excellent treatment. In South Dakota, during the 13 years of 1957 through 1969, 118 communities resorted to ponds for waste treatment (about nine per year). Of these 118 ponds, 54 serve towns with populations of less than 500, 22 serve communities of less than 200, and five serve communities of less than 100. The smallest town served has a population of 87 (1960 census). Of the 118 communities, 48 represent new systems in entirety. In size, the ponds range from a two-cell series of 0.3 of an acre each to a nine-cell complex of 282 acres of water surface serving about 14,000 people and a meat-processing industry (this complex includes three ½-acre anaerobic ponds followed by two parallel series of three facultative ponds each ranging in size from 45 to 47.5 acres each). This large facility provides 100% treatment for 362 days per year, the last two ponds in each series are drawn down for two or three days in early fall to permit complete retention through the late fall, winter, spring, and summer. Although we have not conducted performance studies of this largest of South Dakota installations, *fecal* coliform counts of consistently less than 20 per 100 ml (and occasionally even zero) in the last ponds of each series lead us to believe a satisfactory level of treatment is being obtained (especially since there is no effluent from the system except for two or three days a year!).

The small municipal ponds of 0.3 of an acre each are not unusual and are not the smallest ponds in South Dakota by far. Our latest "head" count shows 29 municipal raw sewage ponds of two acres or less–including ponds in series or parallel operation (95 when Indian schools, missile sites, truck stops, camps, etc., are included). In fact, we are not sure what the minimum possible operational size of a pond may be; we do know our smallest conventional installation is a 60' x 60' pond (0.08 of an acre) serving a truck stop. However, an "unconventional" pond (nine feet deep) in Custer State Park is but 0.065 of an acre in area(3).

As far as minimum possible size of ponds is concerned, we understand that Bill Oswald and his associates, University of California at Berkeley, are using experimental units of about 0.01 of an acre(4). We even hear that Bill currently is developing a portable unit to serve pickup campers and travel trailers. The wheels under these units will have water-lubed bearings, of course. In this way, the tourist will not have to be concerned with the effluent requirements of the various States through which he travels since any overflow will go to the wheels.

There appears to be no upper limit to the size of a community that can be served by ponds, except as economically limited by land costs. The size of any one pond has finite limits, presumably, but, with split flows and parallel operation, the largest of cities could use ponds. Land costs would very quickly become the major and over-riding consideration, however. The vast real estate developments and the urbanization surrounding communities of any size would present some real competition in the land market. In South Dakota, where land for pond sites has gone as high as $525 per acre, with an average of $272 per acre for 32 projects, land costs have not yet become the deciding factor in the choice of treatment method.

As to economics, the pond is still the best buy in treatment facilities. In view of our inflating economy, they might even be considered a good investment! The average per-capita cost of the 118 municipal ponds constructed in South Dakota for the years 1957 through 1959 was $46, including interceptors and, for about one-third, lift stations as well(3). This compares favorably with the statistically projected per-capita cost of about $41 presented by Clare and Weiner in 1960(5).

As you would expect, on an annual basis, costs vary somewhat with average per-capita costs of $31 in 1957, $25 in 1960, $36 in 1962, $42 in 1964, $63 in 1965, $43 in 1968, and $91 in 1969 (the highest on record). The 1969 costs may reflect the anti-inflation policies of the present national administration.

Simplicity of operation has been and continues to be one of the outstanding virtues of the pond method of treatment. Maintenance of dikes to preserve the water-holding capacity of the ponds is probably the *only* absolutely necessary item in operation along with seasonal draw-downs in colder regions to permit complete holding during winter months. We try to insure certain routine care of ponds in South Dakota(6), but ponds continue to function well with or without much attention. It almost seems that a deliberate effort is needed to prevent properly loaded ponds from functioning satisfactorily.

In most South Dakota communities, and in the smaller communities elsewhere in the country we are sure, salaries of municipal employees are not particularly attractive. In most cases, municipal employees are responsible for several functions–dog catching, law enforcement, water supply, garbage collection and disposal, parks and recreation, swimming pools, streets, lighting, etc.

Recently, the South Dakota Municipal League and our Department cooperatively prepared a publication based on questionnaires submitted to 307 incorporated municipalities(7). The questionnaire related to financing, improving, and operating wastewater facilities. Some direct quotes from the publication are indicative of the City Dads' concepts of the need for wastewater facility maintenance and operation. All these communities have wastewater treatment ponds; populations given are from the 1960 census.

Population 139: "Employ one part-time person in water department" (no mention of pond operation).

Population 614: "Employ two full-time employees who work on streets, water works, and sewer departments."

Population 224: "The water department employee is also a policeman" (no mention of wastewater).

Population 875: "Two part-time people spend about 20% of their time in the water department. One part-time person spends about 5% of his time in the wastewater department."

Population 637: "One man handles two jobs in the water department and wastewater department. He is (the) street and water commissioner."

Population 457: "Employ one man. . . who spends 25% of his time in the water department and 10% in the wastewater department. The town pays $410 per month. . .plus a patrol car."

Population 652: "Employ one full-time person who handles the combination job of water, wastewater, street, and parks departments."

Population 495: "Employ two (men) for combined jobs in water and wastewater departments. One works as (the) Superintendent and the other (works) as (the) bookkeeper of both departments."

Population 591: "Three part-time people are employed, with each spending 25% of his time in the water department and 2% in the wastewater department."

Population 850: "Employ one man to maintain streets, water, and wastewater departments. If additional help is necessary, it is hired as needed."

Population 983: "The auditor is the water superintendent, street commissioner, auditorium custodian, police, and cemetery sexton. Occasionally, he does have (the) help of one police officer."

In all of the above circumstances, municipal officials described the arrangements as "satisfactory". I think the point has been made, and the importance of (in fact the *necessity* of) the simplicity of pond operation is established.

Ponds are also useful in prolonging the life of or improving the efficiency of conventional plants. Such ponds receive the effluents from so-called "mechanical" units and provide additional, or "polish", treatment. Five such installations have been constructed in South Dakota and both single ponds and series and/or parallel ponds are used. These polishing ponds have been provided for plant expansion necessitated by increased loadings due to increased populations, industrial wastes, or decreasing efficiency of aging conventional units. What might be called "design slippage" has been involved also (plants that did not provide the degree of treatment for which they were "designed"). In some cases, combined storm and wastewater sewers result in hydraulic overloads which can be tempered with such ponds. In any case, with the evermore restrictive effluent requirements resulting from water quality standards, increasing use of polishing ponds can be expected.

Wastewater ponds have been used for the treatment of, or sometimes the modification of, industrial wastes of a variety of kinds. Without exhausting the industrial applications of ponds, we call attention to ponds for the treatment of food processing wastes, such as slaughterhouse wastes, cannery wastes, sugar beet processing, milk processing wastes, and potato processing wastes, for phenol reduction at oil refineries, for cyanide reduction, for pulp and paper mill wastes, and so on not to mention sedimentation of mining wastes, gravel-washing wastes, etc., which are not the subject of this Symposium.

Our experience with ponds for the treatment of industrial wastes is somewhat limited, a few are used for the treatment of food-processing wastes (anaerobic ponds seem particularly suited to this use, incidentally). Now, we have a number of anaerobic pond systems in South Dakota. Two of them were designed that way. Both of these systems are used to treat packinghouse wastes. In each case, the anaerobic ponds are followed by aerobic units. Since others on the program will discuss the applications of anaerobic ponds, we will hasten on toward the conclusion of this presentation.

Yes, there are problems with ponds. However, the usual spring transition odors are not considered a problem since these odors are an expected characteristic of all but the most lightly-loaded ponds. We have learned, but sometimes forget, that load limits are real; and if we exceed the South Dakota limits of about 20 pounds of carbonaceous B.O.D. per acre, or 50 pounds of nitrogenous B.O.D. per acre we can expect nuisances to increase in direct proportion to the overload *even though overall treatment may be quite good*. It has been demonstrated that even when ponds are rosy-pink with purple sulfur bacteria (Thiorhodaceae), B.O.D. reduction may be satisfactory, and, frequently, odors are essentially absent (8,9,10).

In view of water quality standards and concern with eutrophication, we can expect increased emphasis on chlorination of pond effluents and methods of algae (nutrient) removal. Eventually, because the year 2010 is coming and the U.S. population will have doubled (at least), the pond system may be forced to extinction, like the dodo bird, because there just won't be enough room for them. Until then, the role of ponds in waste treatment is assured.

Having briefly reviewed the role of ponds in wastewater treatment, we would like to add a note in regard to pond behavior.

In the paper he presented at the 1960 Symposium(1), D.F. Smallhorst of Texas also listed seven research needs he thought worthy of attention. Among these was the need "to establish simple control tests on ponds to indicate pond conditions, especially when trouble might be imminent". Now, certain people are said to be able to smell trouble. When a pond is in trouble, anyone can smell it! We are sure Mr. Smallhorst had something in mind other than an olfactory test, however.

In South Dakota, we have a visual "rule-of-thumb" which serves this purpose very well. We call it the "W.I.", or wave index. This index was one of the unpublished fringe benefits of the cooperative Dakota pond studies in 1955-56. While the phenomenon involved may have been noted by some of you, to the best of our knowledge no one has publicly admitted it.

One hot afternoon on one of the overloaded ponds in the study, while rowing the boat with Dr. A.F. ("Fritz") Bartsch riding along, it was noticed that drops of water from the oars, falling onto the surface of the pond, retained their spherical shape, floated on the surface, and were blown across the pond like tiny sailboats. They even showed a V-shaped wake! This was immediately called to Fritz's attention, and we began to speculate on the phenomenon. It was obvious that the surface tension of the pond water - or at least the surface surface-tension - was unusual, and we suspected that the detergents discharged to the pond had something to do with it. Thereupon, we conducted a number of tensiometer tests, but, the results were not conclusive.

Later, at Maddock, North Dakota, during a brisk breeze we noted that while the first pond in the series was hardly riffled, the second pond was beginning to show white-capped waves! A photo of this was published in volume I of the two-volume report of the study(11).

After 10 or 12 years of thinking about this, it occurred to us that this apparent decrease in surface tension was related to loading. Since then, we have observed that the absence of waves on windy days is indicative of an approach to the load limit of the pond (hence, the "wave index") and have coached pond operators to watch for this, particularly where series or parallel operation is possible.

Because of the utility of this index, it has largely replaced our "G.I." (grumper index) and "I.O.I" (inflated-objects index), at least as far as ponds are concerned.

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A DECADE OF CHANGE IN WASTE STABILIZATION LAGOONS IN THE MISSOURI RIVER BASIN

Presented by W. Van Heuvelen, Chief
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The last two decades have seen the development of waste stabilization lagoons as a sole and permanent method of wastewater treatment. Prior to 1950 the use of lagoons as a treatment method was generally discouraged and seldom used in this country. In the early 50's a great upsurge of interest developed in this type of waste treatment. Those directing programs in water pollution in the ten Missouri River Basin states realized a need for developing basic lagoon design criteria, construction and operating practices and were instrumental in developing a symposium on waste stabilization lagoons held here in Kansas City in the first week of August of 1960. All of those involved with this 1960 symposium were amazed at the interest that developed in gathering more information about lagoon utilization. In 1960 a Missouri River Basin Engineering Health Council committee established and published design criteria, construction, and operation practices for the ten-state area.

It is indeed interesting in 1970 to look back at the 1960 lagoon symposium proceedings and the above mentioned committee report and to review some of the information that was developed ten years ago. Most 1960 design criteria have withstood the test of time; however there have been some changes in the use, design and operating practices of the waste stabilization lagoon. This paper will bring out both past and current practices; also some of the changes in the use of waste stabilization lagoons in the Missouri River Basin states.

This paper could not be written by one person speaking for all of the ten Missouri River Basin states. To assist in its development all of the ten Missouri River Basin states responded to an extensive questionnaire. From the interesting results of this survey, a summarization of progress of waste stabilization lagoons in the Missouri River Basin was prepared.

USE

The last decade has seen a tremendous increase in the use of lagoons in the Missouri River Basin. In 1960 there were 469 lagoons while in 1970 there are 1,743 of these installations serving municipalities in the ten states. The breakdown is as follows: Colorado, 100; Iowa 221; Kansas 181; Minnesota, 140; Missouri, 360; Montana, 114; Nebraska, 205; North Dakota, 210; South Dakota, 144; and Wyoming, 68, for a total of 1,743 installations. In addition, there are presently 150 lagoons treating only industrial wastes as compared to 32 such installations in 1960.

Another interesting statistic is that during the past ten years there have been 1,167 new municipal waste collection systems built in the ten states and of these, 851 used the lagoon method of treatment. I believe Mr. Charles Carl, Director of the Division of Sanitary Engineering, South Dakota State Department of Health, brought this out very well in his summary statement which reads: "In the past decade, 78 municipalities have constructed wastewater stabilization ponds in the State of South Dakota. Of the 40 municipalities constructing new sewer collection systems, all the municipalities provided ponds for treatment of their waste. This testifies to the fact that stabilization ponds provided the lowest cost for waste treatment, provides good treatment (sometimes 100 percent) and low expenses for operation of maintenance. We have been very satisfied with this method of waste treatment and will continue to promote this type of facility in the future."

Mr. Mel. Gray, Chief Engineer and Director of Environmental Health Services of Kansas, states: "The use of waste stabilization ponds for waste water treatment increased rapidly between 1960 and 1970, with a total of 26 municipalities using this method in 1960, increasing to 181 in 1970. This does not include the many private installations for single homes, motels, highway restaurants and truck stops, nor for roadside parks and state and federal parks. Objections have been minimal and effluent quality has been satisfactory. During this period 60 percent of the new or enlarged waste water treatment facilities have been waste stabilization ponds."

INDUSTRIAL USE OF LAGOONS

Lagoons are still not used extensively for industrial waste treatment although in recent years their use has increased markedly. In 1960 there were 32 lagoons treating industrial wastes exclusively while in 1970 there are 150. In this survey we discounted small lagoons of two acres or less. There also appeared to be an increased use of aerated lagoons. Mr. Paul Houser, Chief of the Environmental Engineering Service of Iowa, indicated in his summary that "industrial usage of lagoon facilities is also increased, particularly from meat packing and dairy industries. Usually anaerobic and/or aerated lagoons are a part of the industrial lagoon facilities."

LOCATION

During the past ten years there has been little change in attitude as to the location of waste stabilization lagoons. All of the agencies polled indicated they do consider and approve locations from municipal installations. Factors considered include distance to the nearest habitation, distance from the city, wind direction, location of nearby wells, avoidance of surface runoff reaching the lagoon and the accessibility to the receiving streams. In general, the ten states agreed with the statement of ten years ago that waste stabilization lagoons, like other sewage treatment facilities, should be located as far away from existing or future residential and commercial development as is practical and reasonable. Like other sewage treatment facilities, each individual installation merits its own engineering consideration and that location is dependent upon many design factors.

AREA AND LOADING

All of the states agree that loadings and therefore lagoon areas are influenced by many factors including per capita water use, evaporation, rainfall, seepage, degree of treatment desired, growth prospects, waste strength, climate, receiving stream capabilities, and municipal water quality. During the past ten years there was no significant change in requirements or recommendations for design loading. The ten states indicated that the primary cells were loaded in a range from a low of 17.4 pounds to a high of 54 pounds of B.O.D. per acre per day. Some states recommended where a secondary cell is used that it should carry one third of the total design load and others indicated that the loading on the second cell should be the same as the primary cell. Because of the wide range of climates, there also was indicated considerable range in recommended detention time. This time varied from 60 days to 180 days, with a majority of the states recommending 120 days. A few states recommended the use of hydraulic loadings rather than detention time. The returns from the ten states indicated that states with cold winters generally tended to operate facilities to retain all winter time flows. All states indicated that lagoons have demonstrated the capability of handling high loads during the summer months. There has been no change the past ten years in recommendations on the general shape of waste stablization lagoons. Generally the states use round, square or rectangular units with length not exceeding three times the width.

MULTIPLE UNIT INSTALLATION

All of the states in the Basin now encourage multiple cell units. Nine of the states recommend that two cells be constructed and connected in series and/or parallel. One state recommended the two cells be connected in the series only. All indicated they felt the higher degree of treatment could be obtained with the in-series arrangement. The advantage of the in-series and/or parallel is the flexibility provided the operator and the protection it provides a community for adequate waste treatment facilities for future growth. Mr. Jack Smith, Missouri Water Pollution Board, stated it very well when he said, "As the number of lagoons increased, the Missouri Water Pollution Board came to realize that a single cell lagoon was not an entirely acceptable treatment facility due to the large amount of algae which may be and usually is discharged to the receiving stream. As a partial solution, the Missouri Water Pollution Board now requires two, and in some cases, three cells to allow the algae to settle out."

SURFACE RUNOFF-DIKES

All states indicated that lagoons should not receive significant amounts of surface runoff. A few states pointed to this as one of the problems they have encountered with lagoons built in waterways. No significant change in design criteria for dikes was indicated from those listed in the January 21, 1960, Missouri Basin Health Council Committee Report. Two states are requiring, in lagoon specifications, a recommended compaction density of 92 to 95 percent. There is still some variation in dike height but basically it remains at eight feet with inner slopes varying from three to one to five to one and exterior slopes generally at a three to one ratio. Most states still recommend the free board of at least three feet.

LIQUID DEPTH

The survey indicated that optimum liquid depth is still influenced to some extent by lagoon area. Most states recommend an operating depth of three to five feet and a minimum depth of about two feet. The 1960 recommendations indicated a shallow depth should be maintained in the spring and gradually increase during the summer to discourage emerging vegetation. This indicated a spring discharge of waste was desirable. In the 1960 committee report this statement was made: "In the spring the level can be lowered to any desired depth at the time surface runoff and the dilution water are generally at a maximum." Of the ten states recently polled, only two recommended a spring drawdown. All ten states recommended a fall drawdown of the lagoons when the liquid contents had been more completely stabilized. This last point is one of the most significant changes during the past ten years and is a recognition of our increasing awareness to discharge only well-treated effluents to water courses.

LAGOON BOTTOM AND SEEPAGE CONTROL

Most of the recommendations made in 1960 for soil testing, seepage control and lagoon bottom construction still apply. All states recommend soil tests at lagoon sites to decrease seepage problems. Methods of controlling seepage in light soils included clay liners, plastic liners, bentonite, asphalt and polyphosphate applications. One state indicated that prior to approval the state geologist was requested to visit the site to determine if the soil was satisfactory for construction of the treatment facility. The practice of making the lagoon bottom level, free of vegetation and debris, and well compacted with removal of gravel pockets still applies.

CONTROL STRUCTURES AND DISCHARGES

All of the Missouri River Basin states still recommend the placing of the influent line in primary units so the discharge is far enough away from any bank to insure minimum interference with normal circulation. On small installations this means placing the influent line outlet at the center of the lagoon, while in large installations it can be placed 200 to 250 feet away from the nearest dike. All of the states favor building overflow structures which permit operation at selected depths. However there was a considerable variance in opinion as to the level of discharge-either from the surface, near the surface, or near the bottom. The states were about equally divided in this respect. There appeared also to be many different designs on overflow structures and interconnecting piping, ranging from the valved discharge pipe through the dike to the very elaborate discharge control structure. In discussing control structures, Mr. T.A. Filipi, Director of Environmental Health Services for Nebraska, pointed out an important safety factor in lagoon use. He stated, "Furthermore, the lagoon is furnishing a high degree of treatment, and for this reason better quality standards are easily obtained. Lastly, but not least, the bypassing of the treatment facility is practically nil in lagoons because there is no mechanical device that must be taken out of service, overhauled, or replaced. To the best of our knowledge, we have never yet visited one of our 205 lagoons and found bypassing." An attempt was made to find out to what extent lagoon effluents were chlorinated. Three states indicated that they recommended chlorination of some lagoon effluents while two stated they required this practice. Mr. William N. Gahr, Chief Engineer and Director of the Division of Engineering and Sanitation in the Colorado State Department of Health, said that "Our tests on MPN coliform show that 30 days or more of detention would produce a satisfactory dieoff of coliform and we will not normally require disinfection with chlorine." This may be the attitude of many states since only the two required chlorination of effluents.

AERATED PONDS

One of the most interesting developments of lagoon use in the Missouri River Basin has been the increased number of aerated ponds developed since 1960. The survey of the ten Missouri River Basin states showed that none of these installations were in operation in 1960 while at the present time there are 50 municipalities using aerated ponds and 32 aerated ponds treating industrial wastes. Fifty-six of these use mechanical aerators, 22 diffused air, and 12 the oxidation ditch. The survey also showed that 32 of the aerated ponds were followed by additional treatment, showing that at least a third of the ponds were used as a pretreatment before discharge to a conventional lagoon. Mr. Gahr of Colorado made the statement that, "Most engineers are now designing aerated lagoons rather than 'natural' lagoons." Only one state had recommended design standards for aerated ponds. The State of Minnesota used as a design guide a paper entitled "Biological Treatment in Aerated Lagoons" prepared by Donald J. Thimson, former Minnesota State Department of Health Engineer. Aerated ponds seem to be developing a real place in the waste stabilization lagoon treatment picture in the Missouri River Basin.

BENEFITS

In our survey we asked each of the states to indicate if its water pollution control program had benefited from the use of waste stabilization lagoons. We received a unanimous endorsement of lagoons. Mr. A.E. Williamson, Director of the Division of Sanitary Engineering of the Wyoming State Department of Public Health, stated that "The use of the stabilization lagoon was the main factor that allowed the State of Wyoming to accomplish the water pollution abatement program in approximately a nine-year period. The low initial cost allowed construction on a reasonable time schedule and treatment obtained has returned our waters to high quality." Mr. Clay Brinck, Director of the Division of Environmental Sanitation of the Montana State Board of Health, echoed the same sentiment in his summary of waste stabilization lagoon developments in the State of Montana which read "Lagoons have been a most satisfactory means for treatint wastes from Montana cities and towns with populations 10,000 or less. The wastewater stabilization Lagoon has provided this treatment with a minimum of capital investment and operating costs. It made it possible for Montana to achieve the goal of wastewater treatment for all sewered communities in the state by 1967."

Several of the states have indicated that the low initial cost and low maintenance cost were definite advantages in the use of waste stabilization lagoons. This thought was brought out in the summary statement of Mr. Norman Peterson, Director of the Division of Water Supply and Pollution Control of the North Dakota State Department of Health, on the effect of lagoons in North Dakota: "The use of municipal waste stabilization lagoons in North Dakota almost doubled in the past ten years. Today about 86 percent of our municipalities with waste facilities use this method of treatment. A similar increase is noted for use by industries. The waste stabilization lagoon method must be given much of the credit for the excellent progress the state has made in the control of pollution from municipal and industrial waste sources. It has the added advantage of controlled discharge and wastes can be retained during critical steam flow periods."

The Minnesota Pollution Control Agency indicated the use and the value of waste stabilization lagoons for the small community in their summary statement: "The first municipal waste stabilization pond installation in Minnesota was constructed in 1955. Since that time a total of 140 such installations have been constructed serving municipalities with the usual industrial wastes, with populations ranging from 150 to 8,500. Approximately 30 percent of the municipal wastewater treatment facilities in Minnesota are now waste stabilization ponds."

PROBLEMS

We asked the states to list some of the problems they encounter with lagoons. The most frequently mentioned was spring odors during the turnover period. Seven states mentioned that this was a continuing problem. The other next most frequently mentioned problem was the overloading of a lagoon with industrial wastes and the resulting odor problems, not only during the spring but also during the summer and the fall. Miscellaneous problems included seepage through dikes, blocking of drainageways, depth control, dike maintenance, discharge of algae to the streams, and controlling vegetation in the shallow areas of the lagoon. These are problems that are facing a number of installations in the states and perhaps are problems that should be looked at carefully in the period of design of new installations.

SUMMARY

Reviewing the material submitted by the ten Missouri River Basin states the increased use of lagoons as satisfactory waste treatment facilities was forcibly presented. The states pointed out the benefits of low initial cost, low maintenance cost, and the ability to hold waste until the desired time of discharge. The fact that slugs do not affect the treatment in a lagoon and no highly skilled operators were needed was important. A small city could put in an installation which provided good treatment and the fact that there was no shutdown or bypass from the lagoon made this system a dependable method of treatment.

The decade 1960 to 1970 has seen the substantial increased use of waste stabilization lagoons in the Missouri River Basin states. It would appear that the change in their basic design was not as great as that during the previous decade. There is need for a continuing study and research to make them an even better treatment system.

THE ROLE OF OXIDATION PONDS IN IMPROVING ENVIRONMENTAL HEALTH IN DEVELOPING COUNTRIES

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World Health Organization

A small minority of the population of developing countries lives in houses which are sewered. Information collected by WHO in 1962 showed that not over ten percent had piped water in their houses, and only a fraction of those houses are sewered. Where sewers do exist the main reason for building them, especially in rural areas, may have been to recover waste water for irrigating agricultural lands.

Those people who have neither piped water nor sewerage available in their houses make full use of nearby water courses for drinking, cooking, animal watering, washing, and waste disposal. Traditional rules usually prevent a community from contaminating its *own* drinking supplies, but the downstream communities are only protected–if they are far enough away–by natural purifying forces.

To the public health engineer wastes management means, as a minimum, confining wastes at their source and removing them from immediate human surroundings, treating them if necessary to prevent health hazards or nuisances. There are a number of special considerations relating to the health aspects of wastes management in developing countries.

First of all, because of varying customs of personal and community hygiene and because of the greater prevalence of communicable diseases, primary attention must be focused on the way people handle wastes at the source, whether it be the household, the school or the factory. There is still widespread transmission of enteric infections in drinking water and there is the relentless spread of bilharziasis into newly irrigated regions due to bathing and swimming in polluted canals. Yellow fever, malaria and other water-related arthropod-borne diseases have in recent years been controlled by insecticides, but the feeling persists that in the long run only sanitary water management will assure their suppression.

Secondly, as noted earlier, the majority of people in developing countries are not yet in the habit of using water extensively in the household. In some Eastern Mediterranean countries, even after tap water is brought into their homes many prefer to use the public baths and to wash their cooking utensils in an outdoor stream or pool. But their way of life in in transition. Food, clothing, houses, schools, work and leisure are changing, and the amount and kinds of municipal wastes will also change. Once the convenience of a running water supply inside the house is appreciated the domestic use of water and discharge of waste water will augment. How soon and how much are questions the engineer has to struggle with in order to make his designs for ten or twenty years hence.

Thirdly, surface waters in developing countries mostly are not classified as to appropriate usage, and little is known of their quality. Such controls as exist on the discharge of municipal or industrial effluents have been instituted on public health grounds, but they are sometimes so strict that they cannot be effectively imposed in the face of the rapidly growing urban and industrial demands on water use. A lot of research and study is needed.

Fourthly, as already noted, most communities in developing countries, including many large cities, have at present neither facilities for collecting and disposing of waste water nor the financial and organizational means to build and operate them. The problem is by no means simply a technical one–as attested by the large number of handsome engineering reports on sewerage and drainage systems which have not been built. It is, in developing as in developed countries, to present a scheme people want and are willing to pay for. The backlog of needed new utilities construction resulting from recent very rapid urban growth, and competition for development money tend to push sewerage schemes to a low priority.

In the light of the foregoing considerations waste stabilization ponds appear especially suitable for waste water treatment in developing countries. Compared with so-called conventional treatment methods ponds are much cheaper to build, and since they do not require the attention of highly skilled staff they are also cheaper to operate. They can be run successfully with large variations in hydraulic and organic loading. And although they require more space than conventional plants this is often not a problem in developing countries, and in hot climates it is offset to some extent by the possibility of using heavier loading.

Stabilization ponds have already been built in at least thirty-five countries, including over twenty developing countries (Table 1) and are being planned in several others. Information obtained in a WHO survey indicated that although quantitative operational data are not generally available most plants are considered to be operating satisfactorily.

From the public health viewpoint, stabilization ponds may create certain dangers. In limestone areas possibility of seepage into aquifers should be guarded against. Anaerobic ponds, in addition to occasional odour problems, may give rise to fly breeding, if a dry scum forms which is not promptly broken up. Facultative and aerobic ponds may occasionally support mosquito breeding. Snails capable of transmitting bilharziasis have been recovered from an aerobic pond in Uganda. All these vectors represent potentially dangerous situations in countries where the diseases they transmit are prevalent, and point to the need for reliable operation and health department surveillance.

Stabilization ponds may also have an economic disadvantage in dry coastal areas where fresh water is at a premium. In one instance ponds were discarded in the planning stage because the high evaporation from ponds would have significantly diminished the effluent available for irrigation.

We in WHO have been interested in waste stabilization ponds for a number of years, and have endeavored in several ways to stimulate interest and field trials.

The survey to which I referred was made in 1964 with the assistance of Professor E.F. Gloyna, Dean of the College of Engineering of the University of Texas. Professor Gloyna also prepared a paper on waste stabilization pond concepts and experiences which was distributed in 1966 to countries to which the questionnaire had been sent, being addressed both to health ministries and to other agencies responsible for planning and constructing sewerage and waste disposal facilities. The paper was also circulated for comments to a number of persons working in this field. On the basis of replies to the questionnaire and technical information received from the experts consulted, the preparation was undertaken by Professor Gloyna of a WHO monograph on waste stabilization ponds, to serve as a guide to planners, designers and operators, particularly those working in developing countries. Following extensive review by a large number of experts the manuscript has been completed and will be published around the end of this year.

WHO field engineers are assigned in many developing countries to advise governments on the organization of environmental health services and on the development of programmes and projects. In several instances (Nigeria, Ghana, Kenya, Mauritius, Pakistan, and Libya) WHO engineers have advised on the development or on the operation of experimental or full-scale ponds. Research on ponds has been directly assisted by WHO in Argentina, Mexico, Peru, UAR and India.

A number of large pre-investment engineering studies including sewerage and sewage treatment are under way under WHO responsibility and financed by the United Nations Development Programme Special Fund. Here again opportunities present themselves for proposing the use of stabilization ponds, particularly as an interim solution during the early stages of a sewerage construction programme. Other WHO projects are assisting countries to develop institutes and programmes for river pollution study and control. Such work is seriously getting under way in most of the developed countries. A start has been made, with WHO assistance, to survey the situation in a few of the developing countries.

Considering staff training to be one of its most fruitful efforts, WHO has organized a series of short courses and seminars on waste stabilization ponds. In response to the keen interest expressed by engineers in Latin American countries, courses have been held during the past four years in Mexico, Nicaragua, Argentina, Barbados, Bolivia, Columbia, Peru, Cuba, El Salvador, Paraguay, Uruguay, Panama and Ecuador. Similar courses are planned to be held in other regions.

But, as the convening of this symposium shows, the method is still in the midst of extensive experimentation. Ingenious and money-saving ideas are turning up in the laboratory and in the course of engineering studies, whose technical and economic advantages ought to be examined and field tested in different climates. The WHO International Reference Centre for Wastes Disposal, at the Swiss Federal Institute for Water Supply, Sewage Purification and Water Pollution Control in Zurich, has been set up to help meet the need for a continuing exchange of ideas and information in the field of wastes management. By direct contact with over thirty collaborating institutions in all parts of the world, and with WHO's own field programme, the International Reference Centre will, it is expected, contribute positively in this direction.

Great efforts are being made in virtually all developing countries to reach the interim goal of providing safe water in adequate quantities within reach of everyone. This will in turn mean hard work for sewerage engineers and researchers. But their ingenuity and resourcefulness in planning, phasing, designing, and building sewerage systems and treatment plants will quite possibly have the most decisive impact on our much-scrutinized environment.

Table 1. Countries where the use of waste stabilization ponds has been reported, listed by WHO Region

Americas
†Argentina
Bolivia
Brazil
Canada
Colombia
Costa Rica
Cuba
Ecuador
Guatemala
Mexico
Nicaragua
Peru
Trinidad
U.S.A.
Venezuela

Europe
Finland
Federal Republic of Germany
German Democratic Republic
Netherlands
Romania
Sweden
U.S.S.R.

Africa
Ghana
Kenya
Mauritius
Nigeria
South Africa
Southern Rhodesia
Uganda
Zambia

Eastern Mediterranean
Israel
†Pakistan
Saudi ARabia
†United Arab Republic

Southeast Asia
India
†Thailand

Western Pacific
Australia
Japan
New Zealand

† Experimental ponds

DYNAMIC BEHAVIOR OF OXIDATION PONDS

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INTRODUCTION

A stabilization pond can be defined as a shallow man-made basin utilizing natural processes under partially controlled conditions for the reduction of organic matter and the destruction of pathogenic organisms in waste waters.

Initially their design was based on data obtained from ponds in the temperate region of the United States. From research and operational findings in India, South America, Africa and Australia it has become apparent that the behaviour of ponds in tropical and subtropical climates differs significantly from that in a temperate climate, and there has been a gradual and continuous departure from the criteria originally established for ponds in temperate regions.

Stabilization ponds are on the first rung of the ladder of control of biological degradation processes. At the top are processes such as activated sludge, thermophilic completely mixed anaerobic digestion, etc., in which the environmental and physical factors influencing the process are almost completely controlled.

In stabilization ponds the environment is virtually beyond the control of the designer; normally only the physical factors such as depth and loading can be controlled to a degree. As wide fluctuations in the environment are natural the design must be such that the process will operate satisfactorily within a range of expected environmental conditions. Extreme conditions beyond the limits provided for the design will occasionally and invariably occur and the pond will show signs of overloading or fail temporarily.

To provide ponds which will be as effective and economical as possible it is necessary to study the physical, biochemical and environmental factors which influence pond behaviour. Then, within the means available, it can be endeavoured to provide those conditions which will favourably influence the behaviour of the pond. The primary objective is to design ponds that will be of maximum usefulness to the communities they serve.

The nomenclature of stabilization ponds is confusing and sometimes contradictory. For our purpose we shall use the term "stabilization ponds" as a general name for all types of ponds.

Stabilization ponds can be subdivided into the following types:

(1) *Anaerobic pretreatment units*

This sub-division includes ponds in which the whole content is anaerobic, usually termed anaerobic lagoons. It also includes septic tanks, aqua privies and pretreatment tanks-these differ from anaerobic lagoons only in that an artificial cover is provided to shield the anaerobic mass from contact with the atmosphere. In other respects the physical environment and degradation action are identical to those in the anaerobic lagoon.

There is one sub-type: the anaerobic lagoon with an aerobic layer at the surface. These lagoons are usually deep, say about 10 feet or more, with relatively long retention times of say 6 to 9 days. The bottom layers are permanently anaerobic, the top layer is aerobic periodically. The aerobic top layer is reputed to reduce odour development.

All anaerobic pretreatment units have one characteristic in common—their effluent requires further treatment to render it suitable for discharge to receiving dry stream beds, streams and rivers.

(2) *Facultative ponds*

A facultative pond can be defined as one in which (1) the pond liquid content is aerobically-anaerobically stabilized and the major portion of the oxygen necessary for aerobic stabilization is provided by algal photosynthesis, and (2) an anaerobic sludge at the bottom of the pond is stabilized anaerobically.

Oswald[1] defined a facultative pond as one in which "the surface layers are normally aerobic most of the time, whereas its bottom layer is predominantly anaerobic". The situation described in Oswald's definition arises with thermal stratification of the pond contents under conditions of high radiation and low wind velocities. The upper layers of the pond are heated, reducing their density and thereby suppressing mixing with the bottom layers. Oxygen is only generated to the depth the radiation can penetrate. The persistence of this condition adversely affects the growth of the algae and reduces the re-oxygenation capacity of the pond. Where wind speeds are sufficiently high, mixing of the layers of the pond takes place periodically, giving rise to alternating mixing and stratified conditions. The mixing periods promote algal growth and distribute the oxygen through the pond. Generally, mixing exerts a markedly beneficial effect on the behaviour of the pond. Because of the alternating aerobic and anaerobic environment only facultative organisms can exist in the liquid mass of the pond.

Facultative ponds have a distinct sludge layer on the bottom of the pond. This is due to settlement of organic material under the quiescent conditions normally found in the liquid mass above. The internal mass of the sludge layer is permanently anaerobic and the organisms are probably predominantly obligate anaerobes. The sludge layer exerts a signigicant influence on the behaviour of the pond and is probably the most neglected factor in facultative pond theory.

Facultative ponds are by far the most common type of stabilization pond in the world and are often called oxidation ponds. In this paper the two terms will be used interchangeably.

(3) *High rate aerobic lagoons*

In these ponds the depth of liquid is limited to 12 to 18 inches and the retention times from 1-3 days. The shallow depths allow sunlight radiation to penetrate throughout the mass of the liquid. The whole contents, including the settled material, are periodically mixed several times a day by mechanical means. High concentration of algae develops and their photosynthetic action keeps the pond aerobic. The nutrient in the organic pollution is substantially converted to algal cell material which must then be separated from the liquid if the organic content of the effluent is to be reduced—the high rate pond process is essentially a carbon conversion process.

High rate aerobic lagoons require close control and are not suitable for small communities with inadequate technological facilities.

(4) *Maturation ponds*

These are ponds treating effluents from conventional sewage works. They serve to "polish" or "mature" the effluent by reducing settleable solids and faecal organisms; BOD and ammonia are also reduced but to a small extent only.

Generally maturation ponds modify the effluent in such a way that no fungus and filament growths develop in the receiving stream, even if there is little or no stream flow.

(5) *Mechanically assisted ponds*

This type can be subdivided into two sub-types—intrapond and interpond recirculation. In intrapond recirculation the liquid in the pond is recirculated to itself. It has been successfully used in Sweden, where in that cold temperate climate it appears that the objective is to assist in reaeration of the pond contents: the recycled liquid is discharged to a furrow or over packed stones and gravitates back to the pond. If one calculates the daily oxygen requirements of a pond it is soon evident that recirculation cannot supply all the oxygen unless massive recirculation rates are used, as was done in Sweden[2].

However, Abbott[3] has used intrapond recirculation very successfully in South Africa, but at much lower recirculation rates. This poses the question: why has recirculation acted so beneficially on ponds in South Africa? The answer would seem to be that recirculation helps to destroy stratification in the pond and induces better mixing conditions. Adequate mixing during daylight hours induces the nonmotile algae to increase in concentration and provide the oxygen needs through photosynthesis. Thus the aeration aspect of intrapond recycling is of minor importance and attention should be directed to the mixing aspect and the type of installation which will achieve maximum mixing for minimum energy requirements. Intrapond recirculation holds great promise as a means to increase the loading capacity of facultative ponds.

In interpond recirculation liquid is recycled from a secondary pond in a series to the primary pond. In facultative ponds interpond recirculation assists in mixing the primary pond, but its main objective is to reduce the load on the primary pond by "washing out" some of the organic load to the secondary pond. In systems where the primary pond is anaerobic interpond recirculation is principally used to reduce or completely suppress odour development.

By mechanically assisting the pond the next step up the ladder of control of the pond behaviour is taken, in this case by artifically augmenting a condition (mixing) sometimes deficient in nature—it does not therefore replace a condition naturally present.

(6) *Aerated lagoons*

There may be objections to sorting this facility under stabilization ponds, but the theory of facultative ponds can be applied to aerated lagoons with little modification.

One of the main differences between a facultative pond and an aerated lagoon is that in the latter virtually all the oxygen is supplied by surface mechanical or bubble aeration. If the input of energy during aeration is sufficiently high, all the settleable solids in the pond are kept in suspension. Algal growth is absent, or greatly reduced, due to the violent agitation of the liquid and high turbidity. The oxygen concentration in the pond may be quite high (up to 6 mg/l) so that the oxygen transfer rate, which is proportional to the difference between the actual and saturated oxygen concentrations per unit energy input, is relatively low. The input energy is therefore inefficiently utilized for this purpose. Reducing the energy input reduces the oxygen concentration but increases the transfer rate per unit energy input. However, the agitation may now be insufficient to keep all the settleable solids in suspension and a sludge layer is formed on the bottom of the pond. This layer decomposes anaerobically as in the facultative pond.

Aerated lagoons represent a further step up the ladder of control. Here we dispense completely with the natural algal process to supply oxygen and the pond becomes permanently dependent on oxygen introduced by artificial means. Although the rate of introduction of oxygen is now controlled, this benefit is achieved at the cost of increased control.

In the rest of this paper attention will be focused on the behaviour of the oxidation or the facultative pond. Some of the concepts developed will probably be applicable to the other types of stabilization ponds and serve as a basis for clarification of their behaviour.

FACULTATIVE PONDS

Oxidation pond behaviour is determined by a complex interaction of physical and biochemical processes. There is always the possibility that in the detailed analyses of each of the influences acting on the pond the overall perspective is lost. Hence it is desirable to inaugurate the discussion by presenting a simplified picture of the pond behaviour; it is then easier to give the correct assessment and weight to subsequent detailed analyses of each process. A diagram depicting the main biochemical process features of the oxidation pond system, under steady static conditions, is shown in Fig. 1. Essentially we are concerned with the destruction of the energy in the influent; the diagram depicts the magnitude of the flows of energy into and out of each component process by the width of the flow path. The measure of the energy is taken in terms of the 5-day BOD value. This diagram should be kept in mind in the discussion below.

When a waste flow is discharged to a facultative pond a portion of the organic load enters into the liquid body of the pond and the remainder settles on the bottom of the pond to form a sludge layer. In the sludge, the degradation action is anaerobic; in the pond liquid it is mostly aerobic and facultative and, at times, anaerobic.

In the sludge layer, with anaerobic fermentation, the products of fermentation are gases, principally methane, carbon dioxide and nitrogen, and soluble products such as ammonia and other complex organic compounds. The gasses escape to the air, except for a small fraction absorbed by the supernatant liquid; the soluble products are diffused or mixed into the supernatant liquid by the escaping gases.

The gas, methane, has a high energy content (or equivalent BOD content), its loss to the atmosphere therefore reduces the organic load on the whole system. Under constant temperature conditions where steady state equilibrium has been attained the energy lost to the system by the discharge of the gases appears to be of the order of about 30% of the influent BOD. The sludge layer therefore plays an important role in removing pollution from the waste water.

During the early part of the life of the pond the sludge is deposited at a faster rate than the rate at which it can be removed by fermentation, consequently there is a build up of sludge. As the mass of sludge increases the daily quantity lost in fermentation also increases until the mass deposited per day equals the mass lost by fermentation, and an equilibrium sludge mass is established. Depending on the rate of fermentation, which in turn depends on the temperature, equilibrium may be attained only after about 2 to 20 years operation.

The input to the pond supernatant is the non-settling fraction from the influent and the feedback from the sludge fermentation—in this fashion fermentation process is coupled to the liquid supernatant process. If oxygen is present in the supernatant, aerobic degradation takes place. If the pond liquid is mixed the reaction approaches that of a continuously stirred reactor. Provided the environmental conditions (principally temperature) remain constant, an equilibrium BOD value is established in the pond and this is the BOD measured in the effluent. This equilibrium BOD value depends on the influent and effluent flows, influent BOD, settlement of influent BOD as sludge, liquid and sludge degradation rates, BOD feedback from the sludge volume of the pond, mixing conditions, and algal growth.

The degradation rates in the pond liquid and the sludge layer are both temperature dependent. In general, the rates change exponentially with the temperature, but the rate of change with temperature differs markedly between the sludge and the pond degradation processes.

With cyclic changes in temperature one would expect that the BOD in the pond changes correspondingly. For example, the BOD in the pond should be high during winter (because the rate of degradation is low), and low during summer because the rate is high. However the sludge layer considerably moderates this fluctuation. During winter little degradation takes place in the sludge and the sludge mass builds up virtually at the rate of deposition. Hence, there is no feedback to the pond and the BOD in the pond supernatant is determined only by the magnitude of the non-settling fraction of the influent BOD and the pond degradation rate. During summer, when the degradation rate in the pond is high, a low BOD value in the pond is expected. However, the high rate of fermentation in the sludge accumulated during the winter offloads significant quantities of fermentation products to the pond liquid, which raises the pond BOD value. The net result is that generally variation in pond liquid BOD is damped, and with certain yearly cyclic temperature variation virtually no change in BOD can be observed.

Even though the pond BOD may remain sensibly constant, the daily oxygen demand of the pond does not. The pond oxygen demand is equal to the product of the ultimate BOD and the degradation rate in the pond; thus as the pond process rate changes considerably with temperature, the oxygen demand changes accordingly, so that for the same BOD in the pond the oxygen demand per day may be five times or more in the summer than in the winter. Hence it is not possible to gauge the oxygen requirements by inspecting only the pond BOD values. This is one of the reasons why there appears to be little correlation between the pond BOD and the onset of anaerobism in a pond.

The oxygen required for aerobic degradation comes

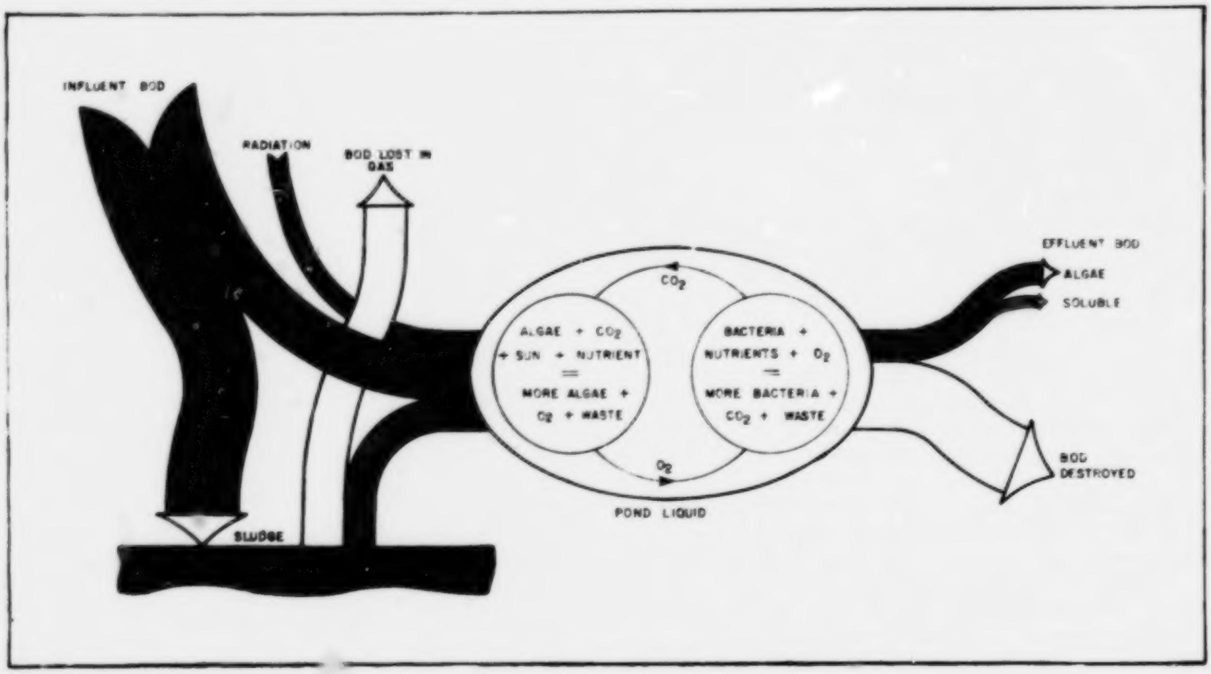
INFLUENT BOD
RADIATION
BOD LOST IN GAS
EFFLUENT BOD
ALGAE
SOLUBLE
CO2
ALGAE + CO2 + SUN + NUTRIENT = MORE ALGAE + O2 + WASTE
BACTERIA + NUTRIENTS + O2 = MORE BACTERIA + CO2 + WASTE
O2
POND LIQUID
BOD DESTROYED
SLUDGE

Fig. 1. Energy flows in Oxidation Pond Degradation [illegible]esses.

almost solely from planktonic algae through the photosynthetic process under the influence of solar radiation. The algae obtain the nutrients necessary for their metabolism from the organic compounds in the pond, and carbon dioxide, necessary for photosynthesis, is obtained from the generation of this gas, aerobic degradation in the pond, and anaerobic degradation in the sludge layer.

Although the algae, by supplying oxygen, are essential to maintain aerobic conditions, this benefit is at the cost of increasing the pollution load for they incorporate solar energy into the chemical compunds of photosynthesis, thereby adding to the pollution load. They also exert a considerable respiration oxygen demand which may completely mask the oxygen demand of the bacteria. Their oxygen consumption is not necessarily directly connected with the demand exerted by the bacterial degradation of the waste material and it is therefore desirable to keep their demand separate from that of the bacteria.

Of the physical factors influencing the behaviour of the pond, mixing is probably the most important. Mixing is induced principally by wind action, although thermal mixing occurs at night to a degree. Lack of wind, coupled with solar radiation normally leads to a state of stratification or non-mixing in the pond.

The intensity of mixing affects the kinetic rate of bacterial degradation in the pond liquid, growth of the algae, their type and concentration. Mixing is also the principal means for distributing the oxygen generated in the top layers to the bottom layers of the pond. Good mixing always increases the capacity of the pond to take increased pollution load. Poor mixing, or stratified conditions, is usually a prerequisite to the development of anaerobic conditions in heavily loaded ponds.

Anaerobic conditions, or "failure" of the pond, implies a qualitative change in the pond behaviour: The pond reoxygenation processes are insufficient to generate aerobic conditions during daylight hours and anaerobic conditions predominate to such a degree that the ecology of the pond is changed; for instance, the algae disappear completely. Re-establishment of aerobic conditions occurs only slowly with time when the offending factors have been removed, or factors favourable to the establishment of aerobic conditions are imposed. As yet it is not possible to determine with any degree of certainty the combination of critical factors resulting in anaerobism.

Mixing conditions

In order to find an experimental basis for the development of a mathematical model depicting pond behaviour, relevant observations over a period of 1¼ years were made on a pond in Lusaka, Zambia, called Matero North Pond. This pond is 5 ft deep and approximately 3 acres in area, treating approximately 0.3m Imperial gallons of domestic waste water per day.

Lusaka is situated inland on the African plateau about 4500 ft above sea level at about latitude 15° South. At this height above sea level the seasons are cooler than the normal for this latitude. Two seasons are recognized: "winter" and "summer", with April and October the transition months. The maximum temperature in summer is about 32°C-38°C and the average minimum in winter is 10°C-14°C. The rainy season occurs in summer, from November to April approximately, and is a period of high humidity. The radiation (Fig. 6C) is remarkably uniform throughout the year, ranging from 400 cal/cm^2 to 600 cal/cm^2, and can be slightly lower in the summer than in the winter owing to cloud cover. The average daily wind speed (Fig. 6A) is higher in winter than in summer; during December and January there is very little wind. The wind blows from approximately 8 a.m. to 11 a.m., then dies away and rises again, but less strongly, during the afternoon.

In Figs. 2A and 4A temperature variations at differnet levels in the 5 ft deep 3-acre pond at Matero North, Lusaka, are shown over a period of a day during winter and summer respectively.

In Figs. 3A and 5A the temperatures on a vertical section at hourly intervals are shown. From these diagrams the following sequence of mixing and stratification can be recognized:

(1) In the morning, if there is sufficient wind, there is a period of mixing indicated by the equality of temperature throughout the depth of the pond. As the solar radiation is absorbed by the pond liquid the temperature rises gradually, the mixing keeping the temperature uniform.

(2) At some time, usually during a short lull in the wind, stratification develops abruptly and a thermocline forms.* (See Fig. 3A). The temperature above the thermocline, the epilimnion, continues to increase to a maximum, usually late in the afternoon, and then decreases as night falls. Below the thermocline, the hypolimnion, the temperature rapidly falls to a value approximately that of the earth,** and thereafter remains practically constant. This is the period of stratification. During this period, if windy conditions persist, a certain amount of mixing takes place above the thermocline, indicated by the uniform temperature down to the thermocline (Fig. 3A), but if the wind speed is low a differential of temperature exists between any two layers throughout the depth of the pond, but particularly the top layers, and no mixing between the layers takes place. (Figs. 4A and 5A).

(3) In the afternoon and evening a second period of mixing may be initiated by one of two mechanisms: (a) thermal mixing or (b) wind mixing:

(a) Above the thermocline, under quiescent wind conditions, the top layers lose their heat by radiation at a more rapid rate than the bottom layers. The cooler top layers sink through the cold lower layers, inducing thermal or convective mixing, with the result that the temperature down to the thermocline remains approximately uniform, but gradually decreasing (Fig. 5A). The thermocline also gradually sinks and, should the temperatures above and below it become equal, with further cooling thermal mixing takes place throughout the pond depth. (Fig. 5A).

(b) Under windy conditions, usually during the period of decreasing temperatures, the energy imparted by the wind to the pond liquid may at some stage overcome the stratification forces and progressively mix the warmer and colder layers adjacent to the thermocline. The thermocline is displaced downwards until the temperature is uniform throughout and the whole pond is again in a state of mixing. (Figs. 2A and 3A).

It is possible that during summer, when wind speeds are low and there is high humidity, cooling by radiation is

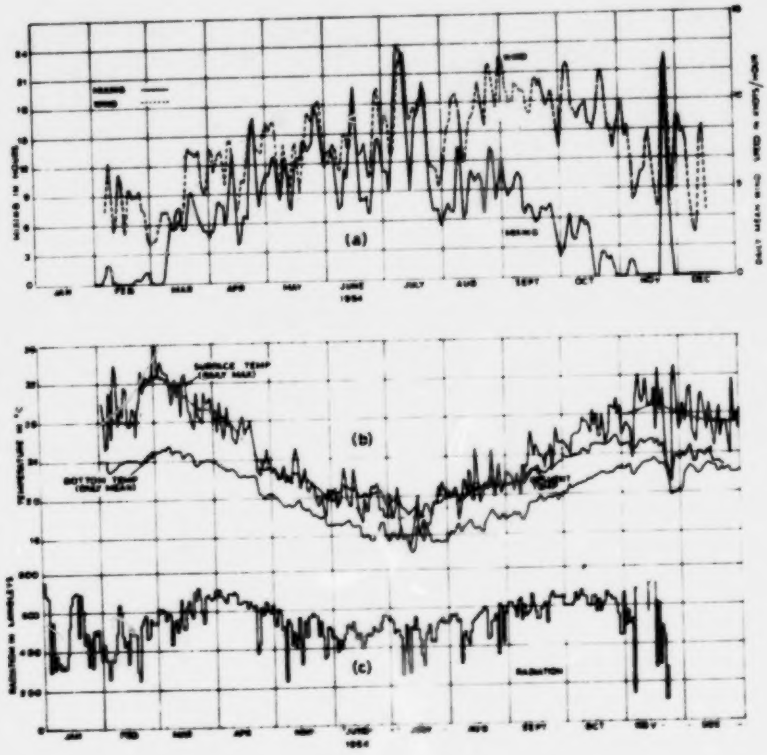

Fig. 6. Wind Speed (a), Mixing (a), Temperature (b) and Radiation (c) at Primary Oxidation Pond at Lusaka over one year.

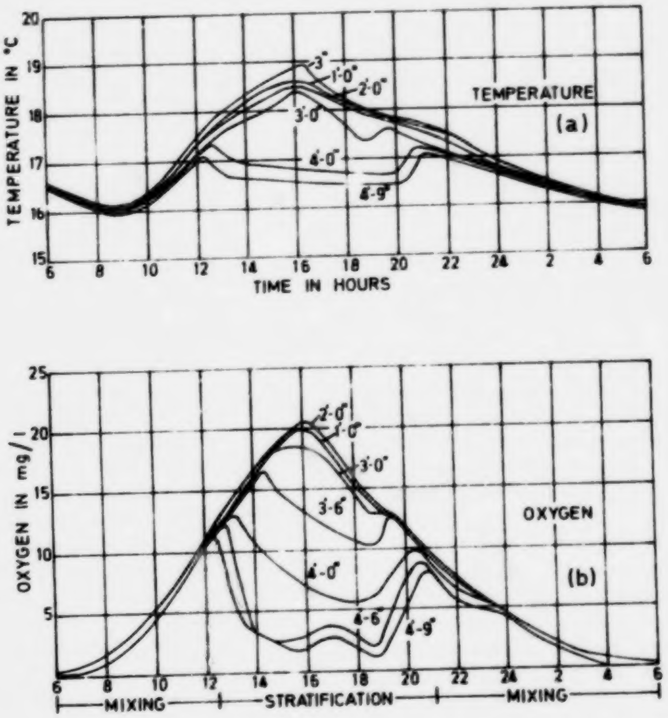

Fig. 2. Variation of Temperature (a) and Oxygen Concentration (b) in 5 ft deep Primary Oxidation Pond at Lusaka during one day under windy winter conditions.

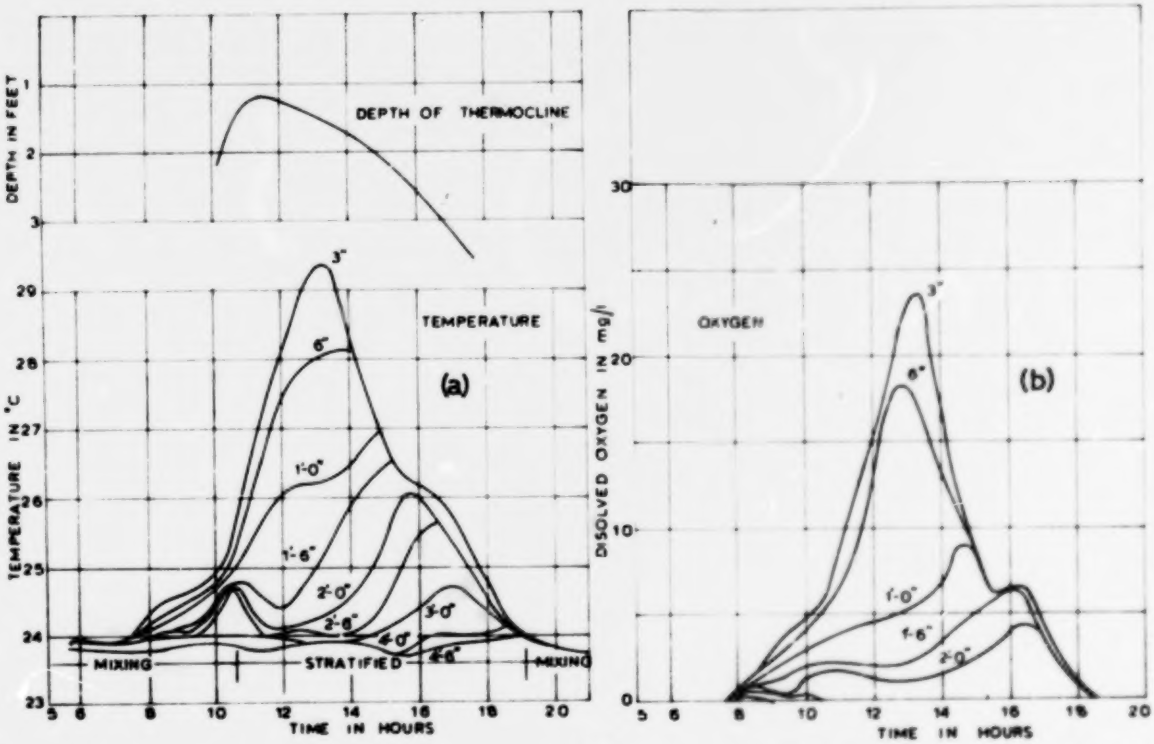

Fig. 4. Variation of Temperature (a) and Oxygen Concentration (b) in 5 ft deep Primary Oxidation Pond at Lusaka during one day under calm summer conditions.

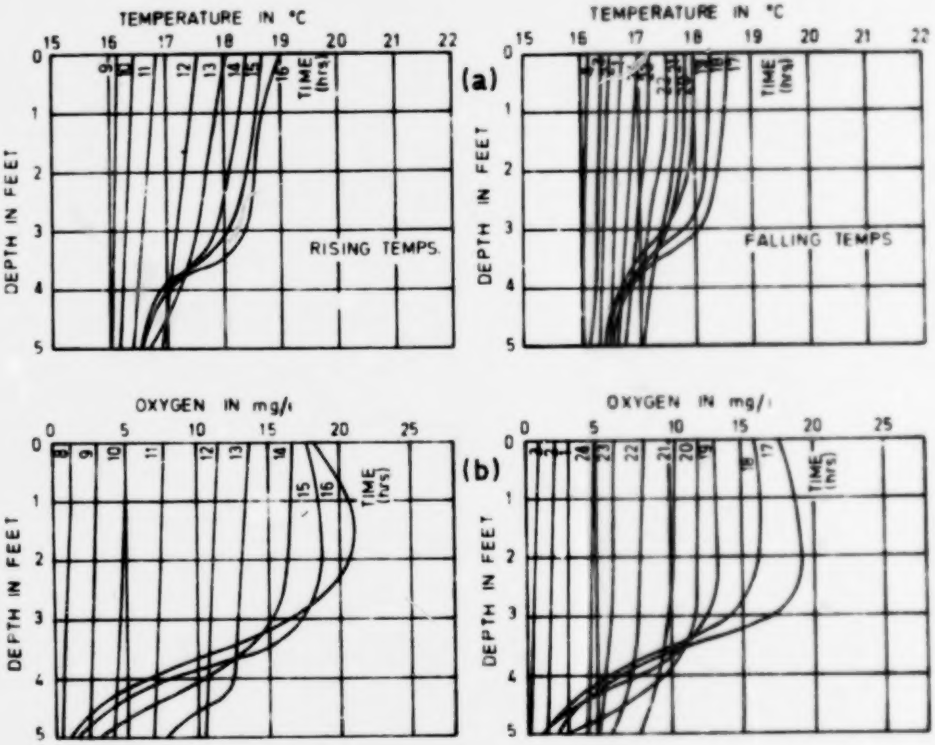

Fig. 3. Temperature (a) and Oxygen (b) Profiles in 5 ft deep Primary Pond at Lusaka under windy wintry conditions.

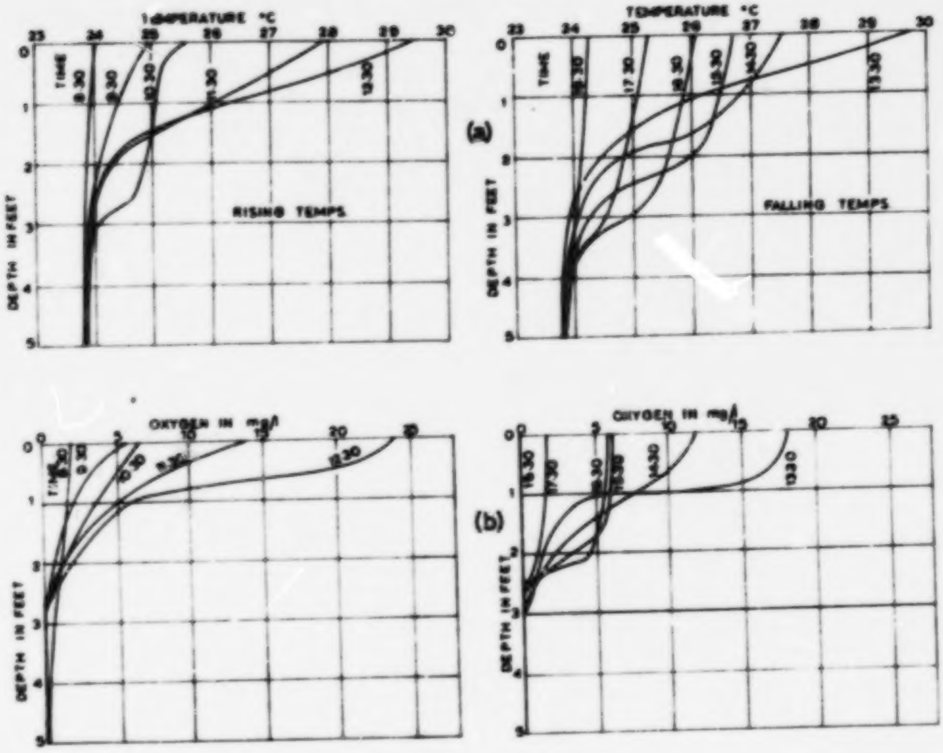

Fig. 5. Temperature (a) and Oxygen (b) Profiles in 5 ft deep Primary Oxidation Pond at Lusaka under calm summer conditions.

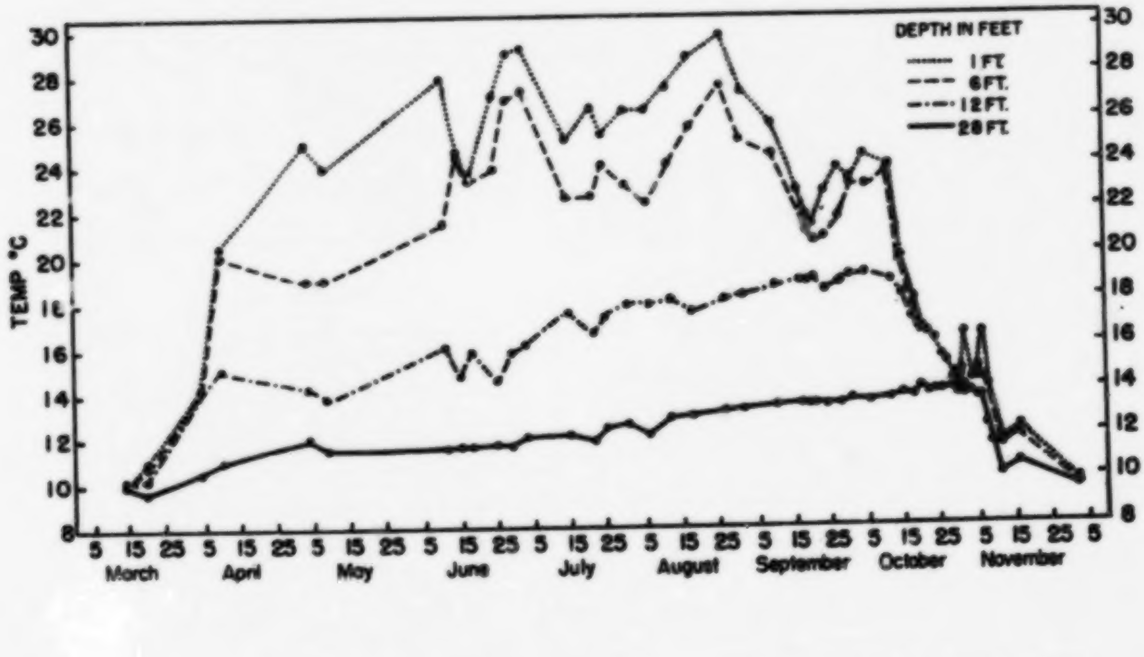

Fig. 9. Temperature variation at selected depth over a year in a storage dam at Chapel Hill, North Carolina, USA.

not sufficient to equalize the temperature throughout the pond and the thermocline persists throughout the period of a day and may continue to do so. At Lusaka, during December 1963 and January 1964, stratification persisted for almost a month and a half.

Over the four seasons the depth of the thermocline oscillates daily—an average depth is about 1 ft during the summer and about 3 ft during the winter. Thus in the subtropics and warm-temperate climates the ponds tend to have a greater volume aerobic during the winter season than during the summer.

At Matero North Pond, where radiation is approximately constant, (Fig. 6), the lengths of the daily periods of stratification and mixing appear to depend principally on two factors: temperature in the pond and wind conditions. The influence of these two factors is illustrated in Fig. 8 by the temperature traces over a period of one day at pond depths of 6 in and 4 ft 6 in for cold, mild and hot days. Initiation and termination points of mixing are indicated by slight jumps on the trace at 4 ft 6 in in depth. The interval wherein the temperature traces at the top and bottom of the pond coincide, defines the mixing period. From trace records over a year, the daily hours of mixing are plotted in Fig. 6A.

Comparison of the average wind speeds, with the hours of mixing, (Fig. 6A) indicates that on windy days the period of mixing is longer than on quiescent days. However, during the warmer months the wind does not appear to be as effective in maintaining mixing conditions as during the colder months. According to Fair and Geyer[4] as the average temperature of the water increases, the wind energy necessary to destroy a 1°C difference in temperature between two layers of water increases significantly. This explains (in Fig. 6A) the decline of the length of the mixing period during the spring and summer, even though the wind speed remained approximately constant.

Depth and surface area

There is a striking similarity between the mixing and stratification cycle in a pond over a period of a day (Fig. 4A), and a lake over a period of a year (Fig. 9). In the shallow pond daily stratification and mixing develop due to the relatively small mass of liquid available to store the heat and the relatively high radiation surface area per unit pond volume. During cooling, convective mixing also brings the warmer bottom layers to the surface. In contrast lakes, due to their greater depth, have a relatively large mass for heat storage, the daily heat gain and loss are confined to the top layers, the layers below the thermocline changing temperature principally by diffusion which is a slow phenomena. Furthermore, the area of surface per unit volume of liquid mass is relatively small. Consequently lakes remain stratified under the prolonged high radiation of summer even under windy conditions and equilization of temperature throughout the lake depth by heat diffusion and mixing only occurs during autumn and winter. Applying these remarks to pond behaviour it can be expected that for the same wind speed, as a pond's depth increases the length of the periods of continuous stratification will tend to increase, particularly during the summer. In deep ponds during stratification, thermal mixing and wind mixing will, of course, always occur in some degree in the upper layers of the pond.

Mixing by wind energy not only depends on the wind speed, but also on the distance the wind is in contact with the water surface (the fetch). The longer the fetch the greater the depth to which the mixing effect extends. Consequently for the same depth, small ponds are more likely to remain stratified than large ones. This is an important consideration when attempting to apply experimental data obtained from small ponds to the behaviour of large ponds.

Algal growth

Of the physical factors which influence the growth, concentration and types of algae in oxidation ponds, it seems that mixing, temperature and radiation are the most important. Their relative importance however changes under different environmental conditions.

Mixing is a critical factor under all environmental conditions: a lack of mixing will affect the algal growth irrespective of the temperature level. Temperature and radiation are normally not cirtical for algal growth in tropical and subtropical regions, but in cold-temperate and sub-arctic regions they may become critical during the winter.

During daylight hours, mixing equalizes temperature and concentrations of oxygen and nutrients throughout the body of the pond. It performs a further vital function by transporting the nonmotile planktonic algae cyclically into the photic zone. In heavily loaded ponds, the photic zone rarely extends further than about 18 in below the pond surface, and often less. Consequently, without mixing, a large proportion of the pond volume remains in continuous darkness.

During long periods of stratification, the nonmotile algae below the thermocline have not the means to enter the photic zone and die off from lack of light. Above the thermocline the diurnal rise in temperature causes a drop in the density and viscosity of the water: (For instance, the viscosity of water at 30°C is 30% less than at 15°C) and this may promote settlement of the nonmotile organisms eventually to sink through the thermocline and die. The nonmotile algae, if trapped for any lengthy period in the top layers by stratification, may also be adversely affected by the high radiation. In general, continuous stratification is characterized by a marked decrease in the concentration of nonmotile algae.

The planktonic algae adapted to conditions of stratification are the motile forms. These organisms migrate and concentrate at those depths of the pond where conditions are most equitable for their existence. For example, during the summer in Zambia, when stratification persists, the ponds appear green in the morning due to the motile *Euglena* moving to the surface, but as the day progresses the ponds turn grey as the *Euglena* move out of the top layer of intense radiation and high temperature to a lower depth, about 1 ft below the surface, and form a dense layer a few inches thick. By thus shielding the lower depths of the pond from the warming effect of radiation, the algal layer promotes the formation of a thermocline at this depth.

During windless days many of the *Euglena* appear to be trapped in the high radiation region. They endeavour to protect themselves by forming cysts and rise to the surface, where they present an appearance as if peppered on to the water. From experimental work on stratified 2 ft deep ponds during the summer in North Carolina, U.S.A., Hartley and Weiss(5) established that: the species *Euglena rostifera* avoid intensities of light in excess of 75 candles per sq ft and migrate to that depth of pond where the intensity is less than this value. In the range of temperatures observed in the ponds, i.e. 21-30°C, the migration does not appear to be influenced by temperature. During the night the *Euglena* migrate to the bottom of the pond into the sludge if flooded with light of high intensity, they encyst.

These observations appear to be in accord with the pattern of behaviour observed in the Matero North Pond, so that the migration pattern appears to apply to other species of *Euglena*, such as *E.polymorpha* Dangeard, *E.viridia* Ehrenberg, and *E.acus* Ehrenberg, the species observed in Matero North Pond.

In Figs. 10 and 11 the algal types and concentrations in two ponds at Lusaka are shown for Matero North Pond (Fig.10) 5 ft, and Matero South Pond (Fig. 11) 3 ft deep. The two ponds are about a mile apart and except for their depths the environmental conditions are identical. At the time the data were collected the BOD5 in the two ponds were approximately the same. During the winter, the algal concentration increased, with peaks of 1.5 x 10^6 per ml, the nonmotile *Micractinius pusillum* Fres. dominating, and at times constituting over 95% of the total algal population. During the summer, the concentration decreased to less than 5.10^4 per ml with the motile *Euglena (E. viridia* Ehrenberg, *E.polymorpha* Dangeard dominant and *E.acus* Ehrenberg frequently) and nonmotile *Chlorella*(sp.) dominating in cycles.

Comparing the hours of daily mixing in Fig 2A with the algal concentration in Fig. 10 it is evident the mixing assisted the nonmotile algal growth, and that stratification during the summer caused the decrease in concentration and the shift of dominance from the nonmotile to the motile types. At no time were the temperatures in the ponds so high that thermal inhibition could be cited as the primary cause for the observed decrease during the summer.*

The favourable influence of mixing is further dramatically illustrated by noting in Figs. 10 and 11 the explosive increase in algal concentrations during November/December, when there was an unseasonable increase in wind speed, causing 24 hours mixing for a few days. (Fig. 6A). In Matero North Pond the *Micractinium* increased, while in Matero South Pond the *Chlorella* multiplied explosively. Whereas in Matero North the increase commenced within a day or so of the period of increased mixing, in Matero South it was delayed a week. By that time the temperature which decreased during the mixing period, had again risen to its former level, thereby providing evidence that the effect was not due to the temperature drop but the after effects of mixing.

With regard to the species of planktonic algae developing in a pond in the warm-temperate climates, it does not appear possible to identify any specific set of causes giving rise to specific algal types: only general tendencies can be identified. Even under identical conditions, the two ponds at Lusaka gave different algal concentrations and algal compositions, as demonstrated by a comparison of Figs. 10 and 11. Hence one may expect very different algal compositions under slightly different environmental conditions. In general, however, the following tendencies appear to be substantiated under tropical to warm-temperate climates. With adequate mixing the nonmotile algae will proliferate and achieve high densities in oxidation ponds. With long periods of stratification the motile alga, the *Euglena* and the nonmotile *Chlorella*** will be the only types that can survive, but their concentrations will be much smaller than the concentrations during mixing.

In cold-temperate regions, where the temperature in the pond liquid during the winter falls below about 12°C, it appears that algal growth is also adversely affected. This situation usually arises during the winter when two factors—low radiation and low temperatures—may be jointly responsible.

The effects of low temperatures, radiation, and wind speeds on algal growth are illustrated in the behaviour of the Fayette ponds in America (Fig. 12A). From September until May the next year the algal concentration was greatly reduced, apparently due to low temperature and radiation effects, even though the wind speed was relatively high and mixing conditions should have been excellent. During May to April conditions were favourable for algal growth—the wind speed was high, temperatures rose to values above 5°C and radiation was up to 500 gm-cal per cm^2 per day. In May the wind speed dropped while the temperature rose, conditions were favourable for stratification, and were reflected by a sharp decrease in algal concentration.

Even where radiation is high, there is reason to believe that low temperatures adversely affect algal growth: In the Highveld regions of South Africa, the temperatures are low in the winter but the radiation remains high and a decrease of algal concentrations is regularly observed during winter. Unfortunately the incidence of stratification during winter is not known.

From the data in Fig. 12 the effects of wind and temperature on mixing growth can be summarised in a diagram shown in Fig. 13. This diagram is an empirical presentation of the interaction between these factors and it should be noted that it is based on the behaviour of two ponds only—Matero North and Fayette. The temperatures here are the mean maximum air temperatures. This temperature is selected as one basic parameter for it seems that the mean maximum pond temperature over say a period of a month very closely approximates to the concomitant temperature of the air (see Fig. 14). The average temperature in the pond does not appear to have this same proportional relationship with the average temperature of the air.

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*In Fig. 12 B the monthly averages of algal densities are plotted for Matero North Pond and it shows clearly the dependence of alga growth on mixing, and its independence of temperature over the temperature range observed in the Matero North ponds.

**The Chlorella appears to be one nonmotile alga able to maintain itself in the pond in appreciable numbers under a wide range of environmental conditions.

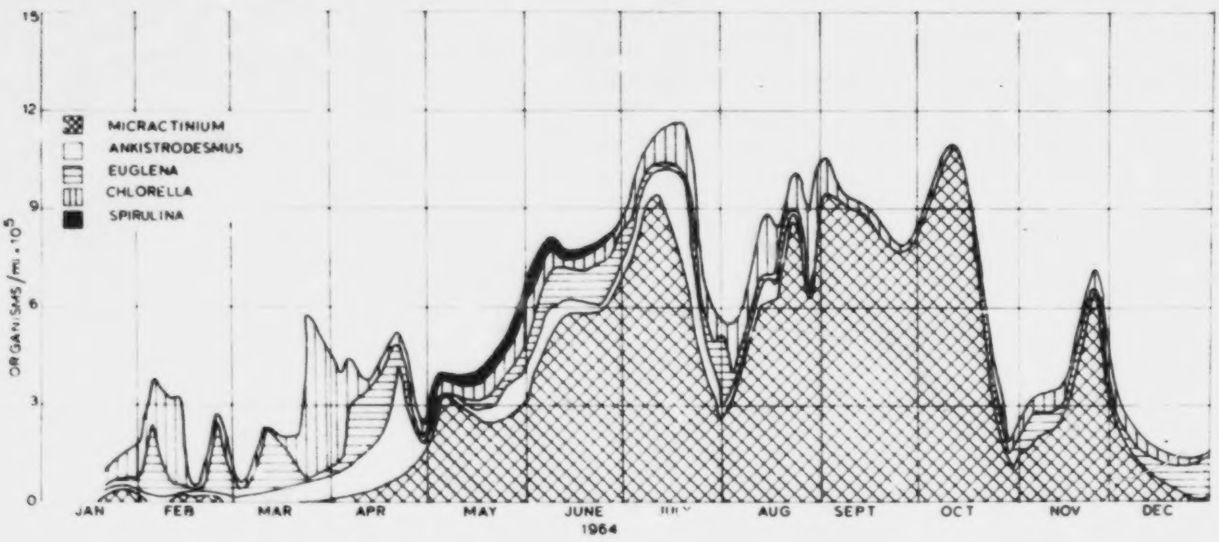


Fig. 10. Concentration of Algae in 5ft-deep Primary Oxidation Pond at Matero North, Lusaka over one year.

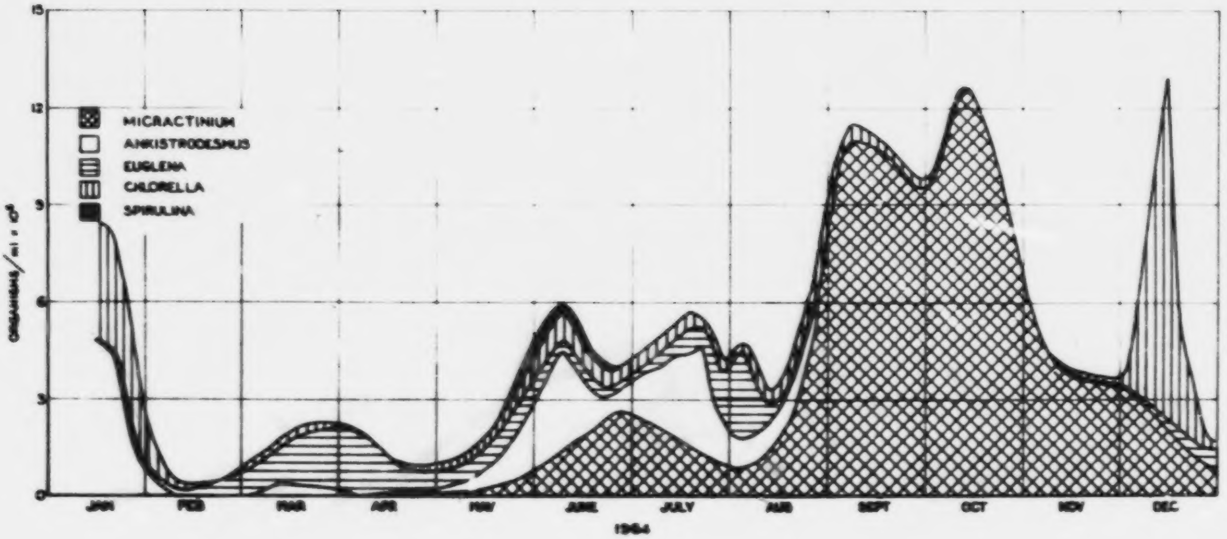


Fig. 11. Concentration of Algae in 3ft-deep Primary Oxidation Pond at Matero South, Lusaka over one year.

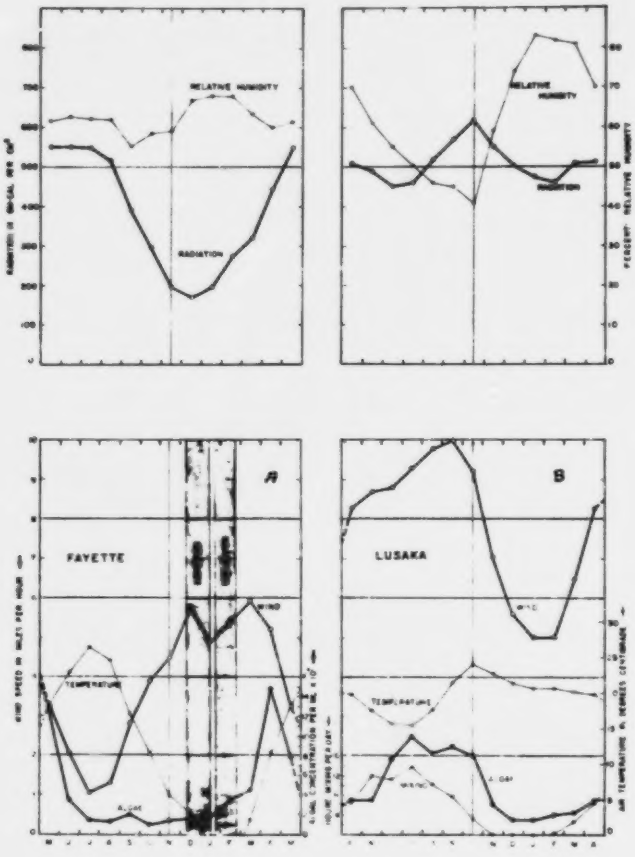


Fig. 12. Comparison of the Behaviour of Matero North Pond and Fayette Pond.

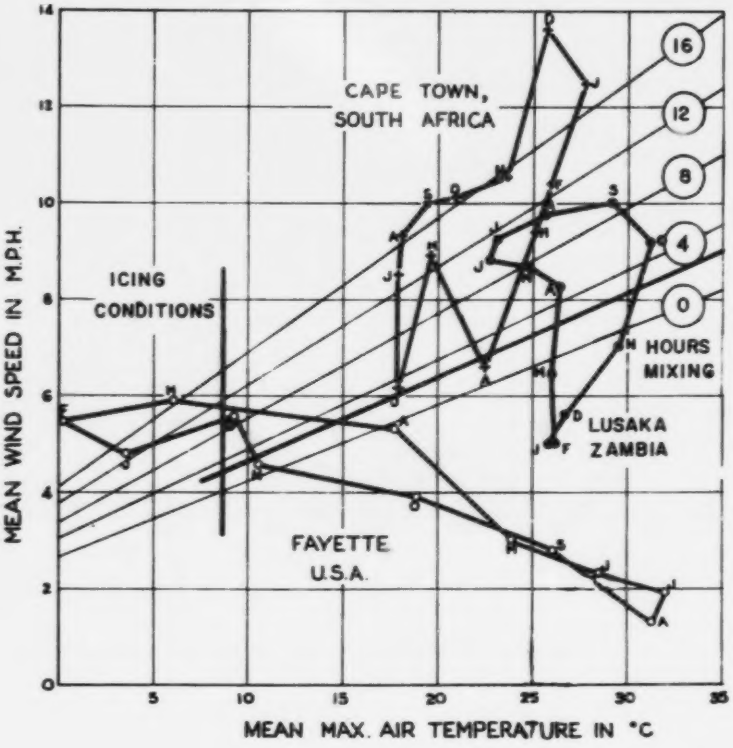


Fig. 13. Influence of temperature and wind speed on pond mixing.

Temperature-wind relationships for the ponds at Matero North, Fayette and Cape Town are plotted on the diagram in Fig. 13, and highlight the difference in environmental conditions between them. The plot can be used to predict approximately the conditions for algal growth in the ponds: Considering the Fayette ponds, the plot suggests that some time during March to April the mixing conditions should be favourable to algal growth. Mixing conditions will become adverse to growth by the beginning of May and the nonmotile algae should decrease. June to October is probably a period of low algal growth due to the lack of mixing, and *Euglena* should become an important constitutent of the algal population. November may or may not be a good month for algal growth. December to February are cold and algal growth will be adversely affected, or disappear entirely when ice covers cut off the radiation into the pond.

Interpreting Matero North Pond, November to March are bad months for algal growth due to the lack of mixing and *Euglena* should predominate. From April to October mixing is good and algal growth should be satisfactory, with nonmotile organisms predominating.

Mixing and aerobic conditions

The influence of mixing on the production of oxygen by the algae is shown in Figs. 2B, 3B, 4B and 5B. The oxygen is dispersed through the pond during the mixing period After the thermocline is established, the oxygen below it is depleted, generally at a high rate because of the high oxygen demand of the anaerobic fermentation products diffusing into the region below the thermocline (hypolimnion) from the sludge layer. Above the thermocline (epilimnion) oxygen concentration increases, at times up to 35 mg/l, i.e. approximately four times supersaturation. On windy days, if the pond is supersaturated with oxygen, the turbulence in the upper layers causes some loss of oxygen to the air, resulting in high concentrations at about 1 ft depth than at the surface. (Fig. 3B). On windless days, if a stone is pitched into the pond with these high oxygen concentrations, the shock causes a cloud of fine oxygen bubbles to rise to the surface.

During windless periods, particularly in summer when the thermocline persists, the oxygen production is confined to the photic zone above the thermocline, and oxygen is only detected to about 1 to 3 ft depth. The region below the thermocline remains anaerobic. Owing to the high rate of degradation once radiation decreases during the afternoon, reduction of oxygen concentration above the thermocline is extremely rapid and the pond may be completely anaerobic for up to 12 hours.

After a cold winter, hot summer weather, with low wind speed and its associated lack of mixing, is particularly adverse to maintain aerobic conditions in a pond: the high temperature induces a high feedback of nutrients from the sludge layer over and above the normal nutrient load in the influent; stratification reduces the algal population and reoxygenation capacity below the oxygen demand of the pond, and the pond may turn anaerobic.

With the objective of direct verification of the effect of mixing on the maintenance of aerobic conditions, a quarter-acre pond at Chingola, Zambia, treating raw sewage was enclosed by a 6 ft high fence with no openings. Within a few days the pond turned anaerobic. When the fence was removed the pond reverted to aerobic conditions.

From these observations the practice of planting trees around ponds to shield the ponds from the public eye, or for beautification, is to be discouraged as it reduces mixing by wind action. Indeed, where the indications are that the ponds are insufficiently mixed or stratified, artificial gentle mixing by mechanical means will destroy the thermocline and promote algal growth and reoxygenation. The power per unit volume required to mix a pond is very small.

Mixing and short-circuiting

Fig. 6B shows the average daily temperatures of the influent and the sludge layer at a depth of 4 ft 6 in together with the maximum temperatures near the pond surface at a depth of 6 in. The daily fluctuation of temperatures of the sludge layer and in the influent rarely exceeds 1°C, but the temperature in the top layer fluctuates between the maximum recorded at the surface and the temperature at the sludge layer.

From Figs. 2B and 8 it is evident that during mild and hot weather the major part of the influent flow enters the pond during the period of stratification, when the influent temperature is lower than the surface temperature. There is, therefore, a tendency for the influent to remain in the lower regions of the pond, but above the sludge layer, as the influent temperature is usually about 2°C higher than the pond bottom temperature (see Fig. 7).

During the winter season the temperature of the influent may remain above the maximum surface temperature of the pond throughout the day (Fig. 8). Even though the mixing period during this season is longer than in summer, streaming and sheet-spreading of the influent on the surface of the pond have been observed, particularly for short retention times.

From these observations, to prevent short-circuiting, it appears that during mild and hot weather a surface draw-off is preferred, while during cold weather a deep subsurface draw-off is preferred. As a compromise for year-round operation, it is suggested that the draw-off should be at aobut 9 in below water level.

To reduce short-circuiting and promote mixing, the inlet to the pond should be near the bottom, discharging in a horizontal direction away from the outlet. Where possible, the outlet should be windward of the inlet.

The sludge layer

The intensity of anaerobic fermentation in the sludge layer can be estimated by measuring the rate of evolution of gas from the layer. Measurement of gas evolution per day from a point in the Matero North Pond, over a period of 9 months, is shown in Fig. 20. In Fig. 21 the logarithm of the rate of gas evolution is plotted against the temperature at the sludge layer, the rate at 15°C being arbitrarily taken as one. The semi-log plot is approximately linear and indicates a marked temperature dependence—there is approximately a sevenfold increase in the rate of gas evolution for a temperature rise of 5°C. A plot of Oswald's[1] observations on a pond in California is also shown in Fig. 21 and the same temperature dependence can be noted.

In the Lusaka pond it was not possible to determine the mass of the fermenting sludge, hence only relative rates of gas evolution can be reported. From visual observation the gas evolution below 15°C is negligible, as the temperature increases above 20°C the rate is so high that sufficient gases are trapped in the sludge to bouy the sludge to the surface where it forms drifting matts. The relative

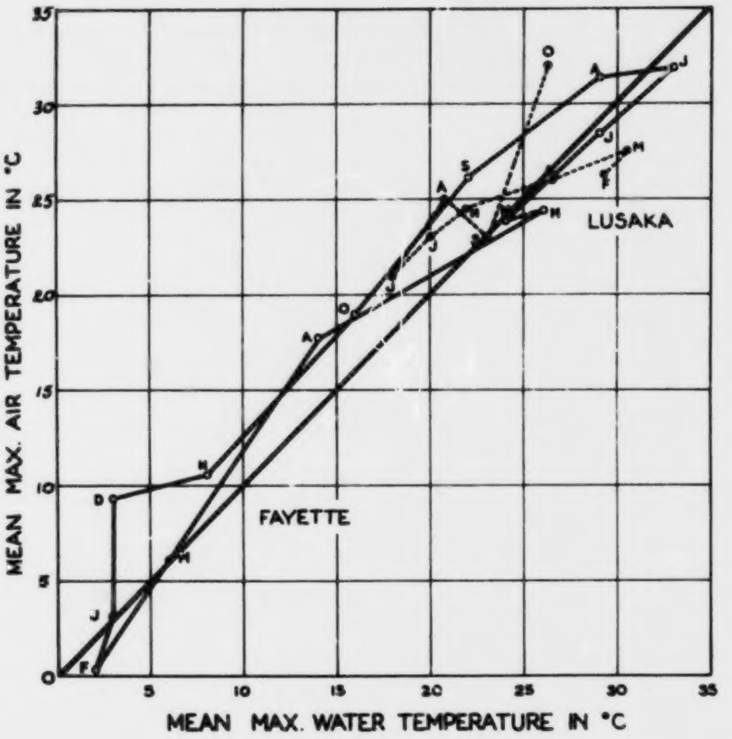


Fig. 14. Relationship between Maximum Mean Temperature in Air and at Pond Surface in Oxidation Ponds.

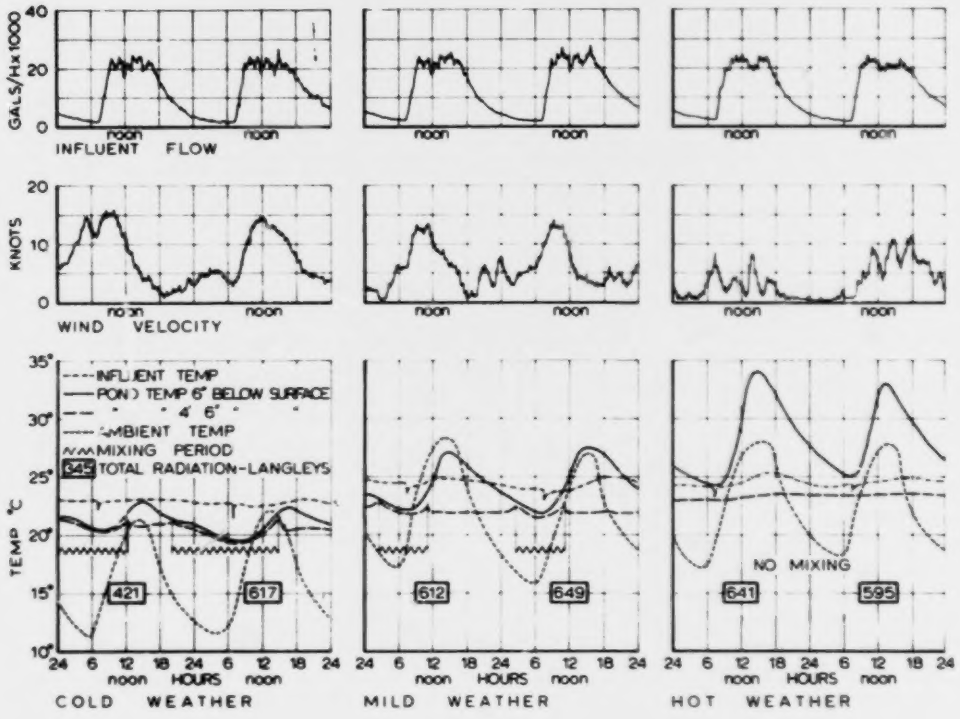


Fig. 8. Comparison of Factors affecting Mixing in Primary Oxidation Pond at Lusaka.

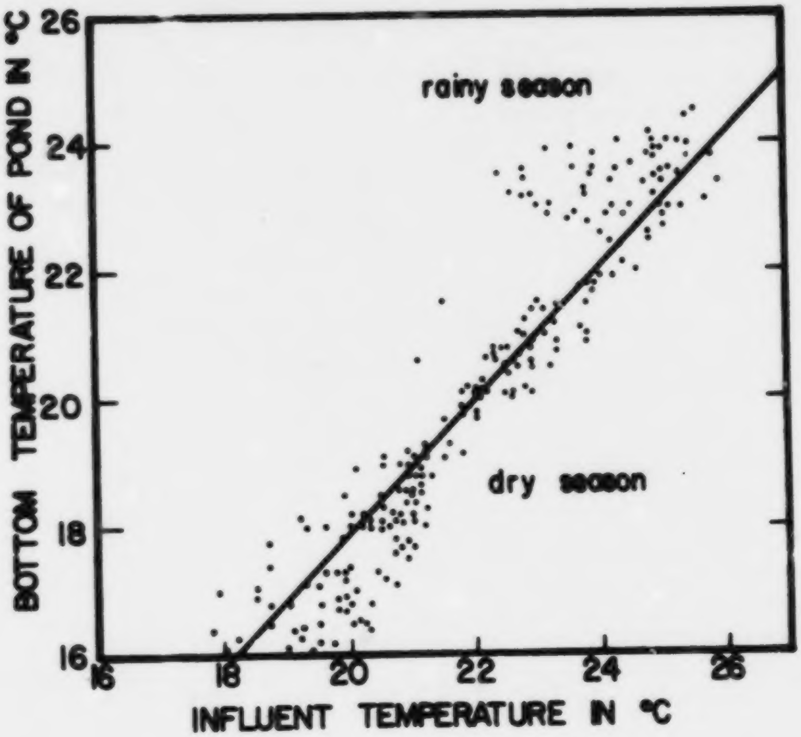


Fig. 7. Relationship between Influent Temperature and Sludge Temperature at Matero North Pond, 5ft deep.

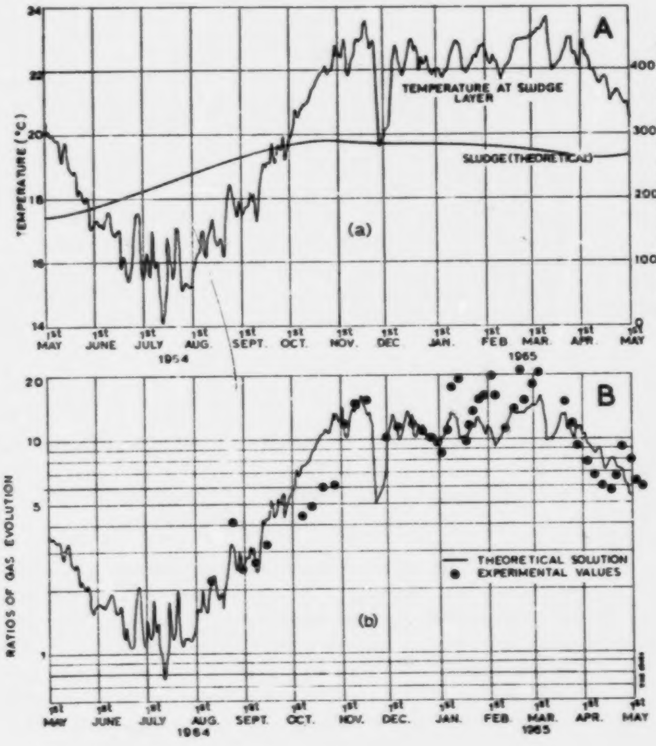


Fig. 20. Sludge accumulation and Temperature (a) in, Gas Evolution (b) from, Sludge Layer of Primary Oxidation Pond at Lusaka over one year.

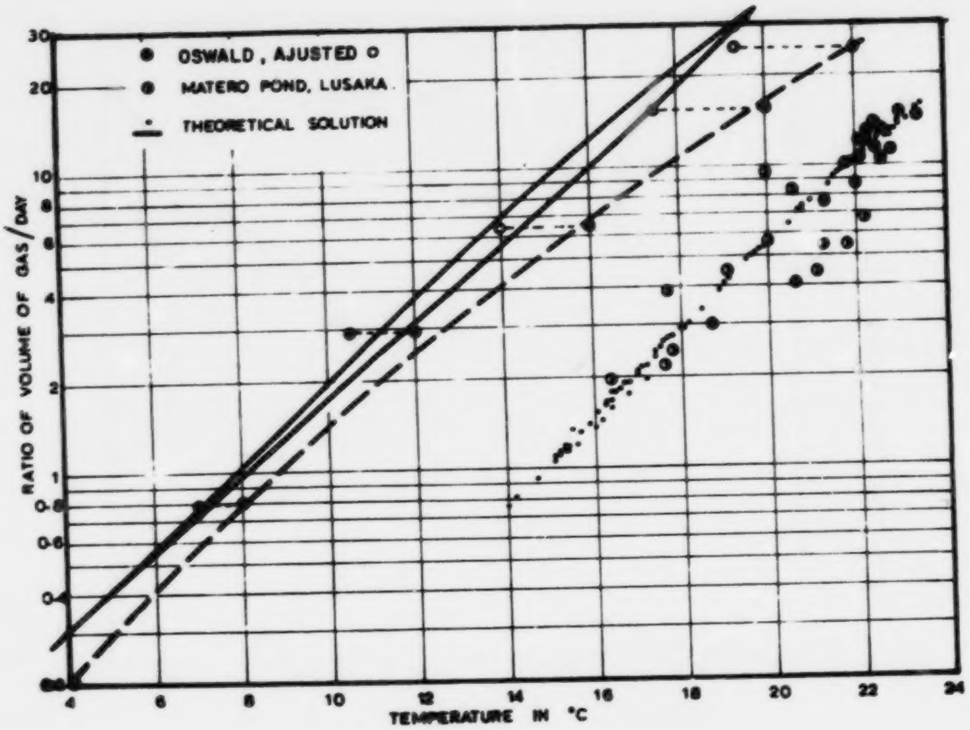


Fig. 21. Experimental and theoretical relationships between Gas Evolution per day from Sludge Layer in Primary Oxidation Pond and Temperature.

area of the pond surface covered by these matts is shown in Fig. 23, which verifies that the critical temperature above which formation from drifting sludge becomes significant at about 23°C.

Sludge settling in an oxidation pond derived from raw sewage influent appears to have a sticky consistency, and the surface matts are only broken up by fairly strong winds. Sludges derived from anaerobically pretreated effluents (i.e. from anaerobic lagoons, septic tanks, aqua-privies and pretreatment tanks) are of a granular consistency and break up on rising to the surface; the trapped gases are released and the sludge particles settle again to the bottom. Consequently little or no drifting sludge is observed on the ponds treating this type of effluent.

Van Eck[6], investigating the performance of anaerobic lagoons treating raw sewage, observed that there is a build-up of the sludge mass during winter and a decrease during summer. Furthermore, the BOD destroyed by the lagoon is higher during summer than during winter. These observations all indicate increased activity of the sludge during summer. During winter the pond bottom is therefore primarily a sludge storage space, whereas during summer the products of fermentation of the sludge accumulated during winter are offloaded to the supernatant pond liquid or discharged to the atmosphere in the form of gas. Consequently the sludge layer relieves the pond liquid of the settleable portion of the influent organic loading during winter, but adds an additional load during summer.

KINETIC MODELS FOR OXIDATION PONDS

Hermann & Gloyna[7], in 1959, presented the first kinetic theory to describe the behaviour of oxidation ponds. On the basis of results from a series of model oxidation ponds, they established that the retention time for the series to give a fixed reduction of BOD_5 of 90 percent at 35°C is approximately 3.5 days. At any other temperature T, the retention time, R_T, was given by

$$\frac{R_T}{R_{35}} = \theta^{(35-T)}$$

where R_{35} = Retention time for 90% reduction at 35° C

R_T = Retention time for 90% reduction at T°C

θ = Arrhenius constant equal to 1.072

Using this equation as a basis, they developed a design procedure as follows: The minimum mean monthly temperature for the year is substituted for T as this gives the largest pond area. The average influent BOD_5 in the United States of America is taken as approximately 200 mg/l, hence 90% reduction gives an effluent quality of approximately 20 mg/l during the coldest month. In order to keep the effluent value approximately constant for any influent BOD5 value, P, the retention time is adjusted in the ratio P/200.

The design method implies that during the summer the BOD5 value of the effluent should be much less than in the winter. Experimental observations on full scale ponds, however, indicate that this is not necessarily the case and that the effluent BOD5 show surprisingly little variation over the seasons.

Marais & Shaw[8] (1961), in Southern Africa, observing the lack of variation of effluent BOD5 over the seasons, proposed a kinetic model based on first order kinetics in a continuously stirred reactor, with the reaction constant independent of temperature. Solving the differential equation for the BOD5 value in the pond [equation (1)] gives equation (3). Marais & Shaw accepted a value for K of about 0.17. Thus specifying P, knowing P_o and K, the retention time, R, is estimated.

By a suitable choice of constants, both theories have found design application in practice. However, a situation in which two theories have such contradictory bases for explaining the same phenomenum is unsatisfactory. Marais[9], in 1965, showed that the two theories are particular instances of a more comprehensive theory. He also indicated that one unexplored facet, the effect of the sludge layer, could possibly explain the reduced fluctuation of effluent BOD values over the seasons.

The rest of this paper will be concerned with the development and verification of the kinetic theory incorporating the effect of the sludge layer and the development of a design procedure. It is however necessary to develop the theory in stages by presenting three models of increasing complexity–each model highlights certain features and in this fashion contributes to the overall understanding of behaviour of oxidation ponds.

Model 1

Assume

(i) Complete and instantaneous mixing of influent with pond contents, hence effluent BOD equals pond BOD.

(ii) Degradation is according to first order reaction with the degradation constant, independent of temperature and retention time.

(iii) No pollution losses due to seepage.

(iv) No settlement of influent BOD as sludge.

Let

P_i = influent 5-day BOD in parts per million (mg/l or ppm) or organism concentration/unit volume (usually ml)

P = pond or effluent 5-day BOD or organism concentration

Q_i,Q_e = influent and effluent flow per day in some chosen unit of volume

V = volume of pond in same volume units as Q_i and Q_e

R_i,R_e = influent and effluent retention times in days defined by V/Q_i and V/Q_e respectively

K = first order degradation constant in $\log_e$ -day units (days^{-1}).

The following differential equation for the reduction process was developed by Marais & Shaw[8]

$$\frac{dP}{dt} = \frac{P_iQ_i}{V} - \left[+K + \frac{Q_e}{V}\right] P \quad \text{or} \; \ldots . (1a)$$

$$\frac{dP}{dt} = \frac{P_i}{R_i} - \left[+K + \frac{1}{R_e}\right] P \quad \ldots\ldots\ldots (1b)$$

Under equilibrium conditions with P_i, R_i and R_e constant

$$\frac{dP}{dt} = 0 \text{ and}$$

$$P = \frac{P_i}{KR_i + \frac{R_i}{R_e}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

If seepage and evaporation losses are neglected $R_e = R_i$ and equation (2) reduces to

$$P = \frac{Pi}{KR_i + 1}$$

The ratio Ri/Re ranges from zero to one, depending on liquid losses, but can be taken as equal to one with little error. In short retention ponds losses will be small so that the ratio will be near 1, and in long retention ponds KR_i is very much larger than Ri/Re so that the influence of the latter is small. For simplicity, Ri is replaced by R in equation (3) above.

In a series of ponds let

R_n = retention time of the nth pond

P_n = effluent BOD from the nth pond.

Applying equation (3) sequentially to each pond in the series

$$P_1 = \frac{P_i}{KR_1 + 1}$$

$$P_2 = \frac{P_i}{KR_2 + 1} = \frac{P_i}{(KR_2 + 1)(KR_1 + 1)} \text{ and so on.}$$

Hence

$$P_r = \frac{P_i}{\prod_{n=1}^{r} (KR_r + 1)} \quad \ldots . (4)$$

If $R_1 = R_2 = \ldots . R_n = R$

$$P_n = \frac{P_i}{(KR+1)^n} \quad \ldots . (5)$$

If the total retention time remains constant nR = constant = t (say), hence R = t/n and substituting in equation (5)

$$\lim_{n \to \infty} P = P_i \, e^{-Kt} \quad \ldots . (6)$$

which is the plug batch, or river flow condition.

Equations (3) and (5) have been successfully used in Southern Africa to predict the reduction of faecal bacterial pollution. In Fig. 15 the average reduction of *E.coli* type 1 in single ponds with different retention times, and two series of ponds are shown. The plots indicate conformity with equations (3) and (5) with the value of K = 2 approximately. It is evident that for the same total retention time the reduction in a series of ponds is significantly superior to that in a single pond. For this reason series of ponds are generally used to reduce bacterial concentrations in sewage works effluents in Southern Africa.

No observable dependence of K on temperature and retention time can be noted: Regarding temperature, the location of the ponds, shown in the plot in Fig. 15, ranges from the temperate region in Johannesburg, South Africa, to subtropical Zambia so that a significant temperature range is covered. Regarding retention time, plots of the average values of K against retention time for a number of ponds for *E.coli*, *F.strep.* and Total Coliform are shown in Fig. 16. The values are randomly distributed about the geometric mean value of 2, but there is no evidence of dependence on retention time.

Probability cumulative distribution plots of the K values for the three organisms show that the distributions are lognormal with geometric mean values around 2. (Fig. 17). Statistically the means and standard deviations for the organisms are not significantly different, so that the consolidated geometric mean value can be accepted as the expected value, very nearly 2.0.

The observation that the data appear to be random lognormally distributed indicates that no assignable cause can be given for the distribution of K values: That is, the values are due to innumerable influences acting in random combination on the ponds. Thus it is possible that two identical ponds can have geometric mean (expected) values for K of 0.3 and 3.0 respectively without short-circuiting necessarily occuring in the pond with the lower value of K.

A combination of ponds in series is more likely to give an effluent near the expected value: The values of K are random "selections" from the lognormal population of K and the average effect on the effluent from a series will therefore tend to approach the reduction determined on the basis of the expected value of K = 2.

Where quantitive changes in the pond behaviour become significant, the expected value of K may change correspondingly. For example, in very cold climates the indications are that K decreases. With qualitative changes, for example under anaerobic conditions in septic tanks the value of K is also reduced. In Zambia tests on the influent and effluent concentrations of *E.coli* in septic tanks with retention times of about one day show little reduction, i.e. the *E.coli* does not appear to find anaerobic conditions as adverse as aerobic conditions. For this reason also during summer, with low wind speeds, when stratification causes the lower depths of the pond to be more or less permanently anaerobic, significant reduction in the value of K occurs. This effect is illustrated in Fig. 18 which shows the distribution of influent and effluent *E.coli* during winter and summer periods – the value of K derived from the mean reduction during summer is significantly lower than during winter.

Model 2

Model 2 is identical to Model 1, except for assumption 2, which must be modified as follows:

(2) Degradation is a first order reaction with the reaction rate dependent on temperature according to Arrhenius's relationship. (Equation 8).

Under steady conditions of influent flow BOD and temperature equation (1) applies and all its consequences. If the temperature changes and effluent BOD_t is given by

$$P = \frac{P_i}{K_T R + 1} \qquad \ldots.(7)$$

where $$K_T = K_{T_o}\,\Theta^{-(T_o - T)} \qquad \ldots.(8)$$

Equation (8) is Arrhenius's equation relating the value of K at temperature T with K_{T_o} at Temperature T_o.

These equations apply to the BOD reduction in a pond when no settlement takes place. Suwannakarn[10] did a number of tests on a series of six laboratory scale oxidation ponds ($V_1:V_2:V_3:V_4:V_5:V_6$ = 1.4:1.0:1:4:1:4:1.0:1.4) using a non-settling milk waste. Each test in the series was done at a different temperature and two inflow rates were used. Applying the series solution [equation (4)] and equation (8) it is found that values $\Theta = 1.085$, $K_{T_o} = 1.2$ at $T_o = 35°C$ gives the best overall fit. Correlation between the theoretical and experimental data for each pond in the series is shown in Fig. 19.

This model and its verification is of great importance in the development of pond kinetics. It establishes the kinetic behaviour of the liquid fraction of the pond independent of the sludge layer. It also allows integration of Hermann & Gloyna's[7] and Marais & Sahw's[8] theories as follows:
From the symmetry of R and K in equation (7) it can easily be shown

$$\frac{R_{T_o}}{R_T} = \frac{K_T}{K_{T_o}} = \Theta^{-(T_o - T)} \qquad \ldots.(9)$$

With K = 1.2 at $T_o = 35°C$ from equation (7) for 90% reduction (10% remaining) $R_{35} = 7.5$ days. Hermann & Gloyna give a value of $R_{35} = 3.5$ days. The difference is due to the fact that they determined the total retention time for the series of six ponds at specified volumetric ratios to give 90% reduction. The constant, 3.5, is in fact not applicable if different volumetric ratios or a different number of ponds in series are used.

Model 3

This model incorporates the influence of the sludge layer. An expression for the degradation occurring in the sludge layer can be incorporated in the differential equation if it is assumed that anaerobic degradation of the sludge, which manifest itself in the evolution of gas and release of fermentation products, is a first-order reaction. Further assumptions are:

(1) All BOD's are ultimate first stage. Although not essential, this assumption simplifies to a degree the subsequent formulations: for example, calculation of the oxygen demand rate is simplified. Furthermore, the 5-day value depends os the temperature and the technique of testing so that for consistent results these must be strictly standardized. In contrast, according to Gotaas [11] the ultimate BOD value is relatively insensitive to temperature. Unfortunately the determination of the ultimate BOD is time consuming at present, but as shorter methods are developed based on oxygen consumption rate determinations [12] ultimate BOD values will find inceasing application. Normally, however, only the 5-day value is available and the ultimate value must be estimated by multiplying the 5-day value by a constant, which is known from experience or determined by a special series of tests. For example, this constant is approximately 1.47 for domestic wastes. If such an estimation is unreliable experimental process models must be constructed. This is the normal procedure for industrial wastes.

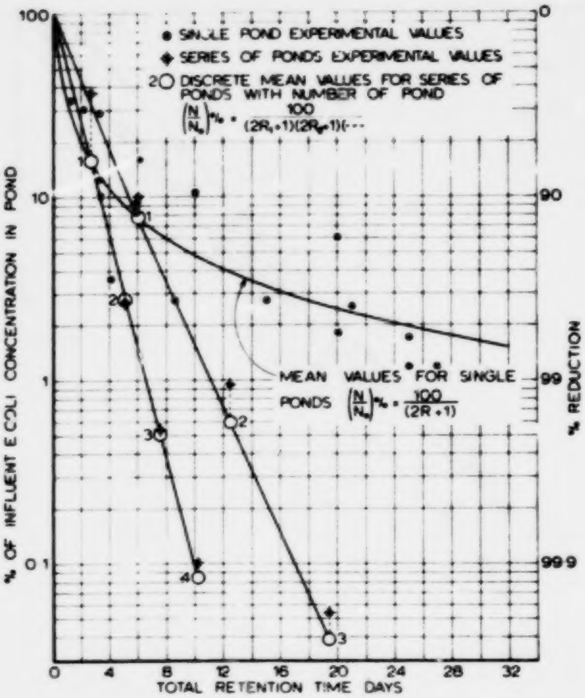


FIG. 15. Experimental and Theoretical Concentrations of E.coli in single pond for different retention times and in series of ponds, for K = 2.0

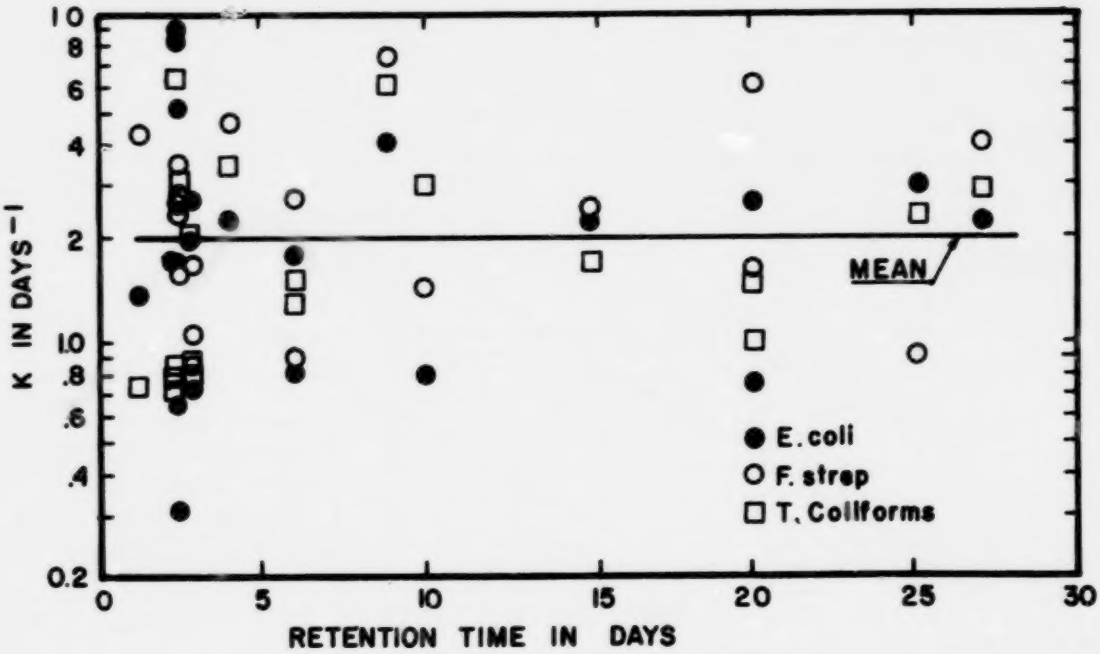


FIG. 16. Variation of K values with retention time for a number of ponds.

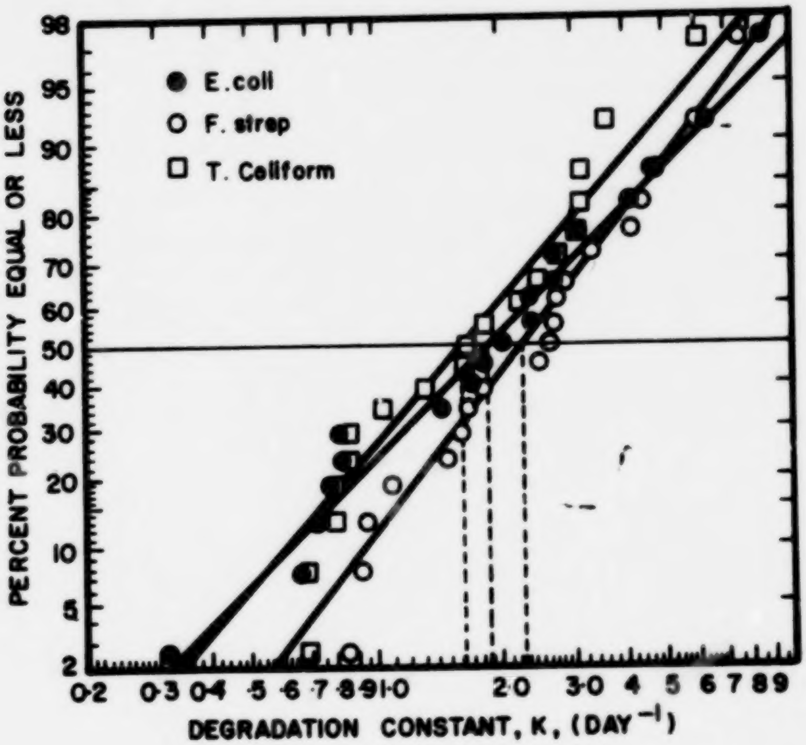


FIG. 17. Statistical distribution diagrams of K values for E.Coli, F.strep and Total Coliform for a number of ponds in Southern Africa.

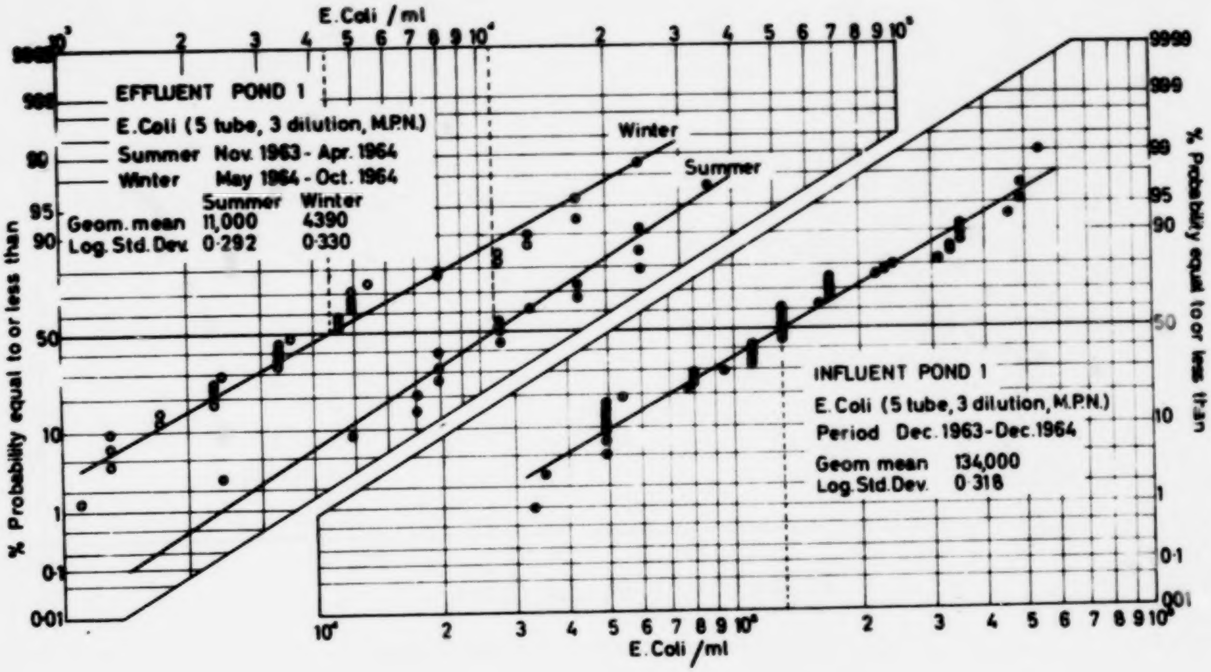


FIG. 18. Seasonal change in Concentration of E.coli in effluent from Primary Oxidation Pond at Lusaka.

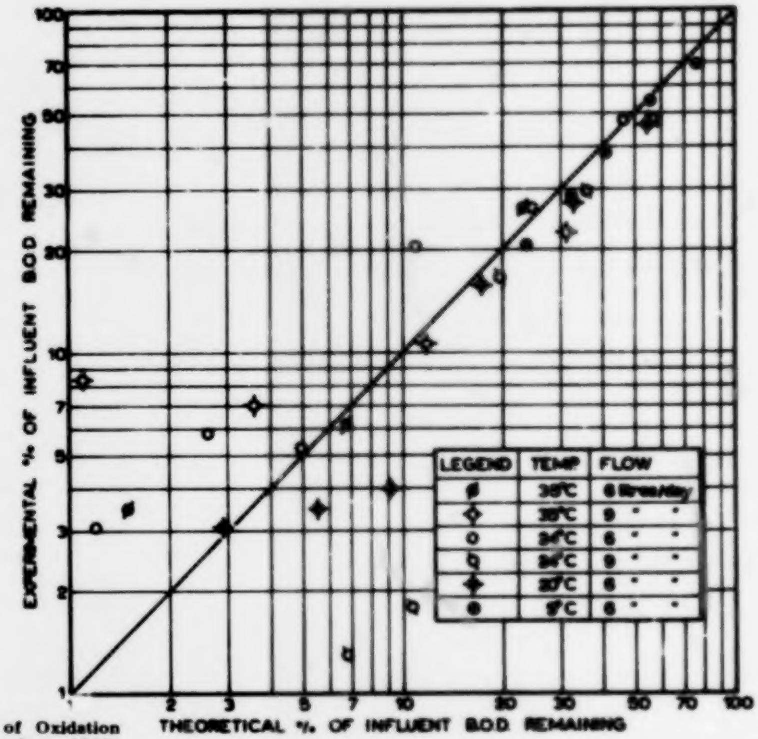


FIG. 19 BOD removal in laboratory-scale series of Oxidation Ponds: Correlation of experimental (Suwannakarn, 1963) and theoretical results.

(2) A fraction, i_p of the influent ultimate BOD, P_{ui}, is dispersed within the liquid body of the pond, the remaining fraction, i_s, settles as a sludge.

(3) There is complete and instantaneous mixing in the pond, hence the effluent BOD is equal to the pond BOD. At first sight this may seem an unwarranted assumption, but previously in this paper it was demonstrated that ponds normally have daily periods of mixing; even in ponds with retention times as short as 10 days, an influent slug of pollution is therefore gently mixed about 10 times in the pond before discharge. Relative to the retention time, mixing is therefore quite thorough.

(4) The process reaction constant, K, for the pond liquid, and K_s for the sludge layer are temperature dependent according to the following relationship:

$$K_T = K_{T_0}\, e^{-c(T_0 - T)} \qquad \ldots.(10)$$

with $\Theta = e^c$.,

(5) A fraction, s_p, of the BOD lost from the sludge due to fermentation enters the pond liquid volume; the remaining fraction, s_g, leaves the system as gas.

The following differential equations describing the pond model can be derived:

(1) Rate of change of sludge mass, St

$$\frac{dS_t}{dt} = -K_s S_t + i_s P_{ui}(\Upsilon Q)\,.10^{-6} \qquad \ldots.(11)$$

where

K_s = temperature-dependent according to equation (10)

S_t = total mass of sludge BOD in lb or kilogram

Υ = mass per unit volume of pond liquid (1Kgm/m^3, 62.4 lb/ft^3, 10lb/Imp gal).

Q = daily flow in M^3, ft^3, gal.

(2) Rate of gas evolution from sludge

$$\frac{dV_g}{dt} = C_v\, s_g\, K_s\, S_t \qquad \ldots.(12)$$

where C_v = volume of gas liberated per unit mass BOD destroyed.

(3) Rate of change of BOD, P_u, in the pond. Change of BOD in time, dt, equals

(a) Reduction due to degradation action in the pond and loss in effluent.

(b) Increase due to influent fraction, i_p, to body of pond and fermentation product

$$\frac{dP_u}{dt} = -\left[K + \frac{Q_e}{V}\right]P_u + \frac{i_p\, P_{ui} Q_i}{V} + \frac{s_p\, K_s S_t}{V\ \Upsilon}.10^6 \quad \ldots.(13)$$

If the influent BOD, flow rate and temperature remain constant, noting that $s_p + s_g = 1$ then at equilibrium dS_t/dt = O and dP/at = O and equation (12) reduces to

$$S_t = i_s\ P_{ui}\ Q/K_s \qquad \ldots.(14)$$

Furthermore if $Q_e = Q_i$ so that $R_i = R_e = R$ equation (13) reduces to

$$P_u = \frac{P_{ui}}{KR + 1} \quad (i_p + s_p i_p) \qquad \ldots.(15)$$

Thus under steady state conditions equation (15) describes the relative magnitude of effects of the influent non-settleable and the sludge fermentation BOD's on the pond BOD. Equation (15) is, however, of little practical value as temperatures fluctuate seasonally.

Rate of oxygen utilization is found as follows: A sample of pond liquid has an oxygen consumption-time relationship

$$Y = P_u\ (1 - e^{-Kt}) + K^1 t$$

where K^1 = oxygen
consumption rate constant due to the algae
t = time in days

The rate of oxygen utilization, r, is given by

$$r = \frac{dY}{dt} = P_u\ K e^{-Kt} + K' \text{ mg/l/day} \quad \ldots.(16)$$

At t = O equation (16) gives the rate at any time zero, i.e. at the time the sample was taken:

$$r = (P_u K + K^1) \text{ mg/l or (ppm)/day} \qquad \ldots(17)$$

Total oxygen consumption for the pond per day

$$= \frac{rVT}{10^6} \qquad \text{lb or kg} \qquad \ldots.(18)$$

where V = volume of pond in gallons or m^3
T= mass density (lb/gal. kg/m^3)

Evaluation constants

Very little information is available on the magnitudes of the fractions i_p, i_s, s_p and s_g. Judging from the behaviour of settling tanks the fraction of influent BOD settling as sludge, i_s, is probably greater than 0.4, and considering the long retention times in ponds and the generally quiescent conditions in them, a value of 0.6 is probably nearer reality. Considering s_p and s_g, the fraction of sludge fermentation products to the pond liquid and escaping as gas respectively, nothing of a quantitative nature is known. After many trial runs of solutions of equations (11) and (13) on a computer, the best fit to experimental pond BOD data appears to be given with $s_p = 0.4$, hence $s_g = 0.6$. The value of C_v is taken as approximately 10 cubic feet of gas for every pound of sludge BOD destroyed as gas.

To determine K_{so} and Θ_s equations (11) and (12) show that for any BOD loading the *ratios* of gas evolution at different temperatures can be determined theoretically without knowing i_s or s_g provided these fractions remain constant. From Fig. 21 the experimental relationship between the logarithm of the ratios of the rates of gas evolution and the temperature is approximately linear; from trial solutions of equations (11) and (12) it became apparent that such linearity is only possible if C in equation (10) is large. The slope of the line in Fig. 20 is then approximately given by C $\log_{10}e$, hence an initial value of C is found. Using the daily temperatures measured in the sludge and integrating equations (11) and (12) numerically on a computer, the best fit was obtained with the values $K_{s0} = K_{20} = 0.002$ and $C = 0.3$ at 20°C in equation (10), i.e. $K_s = 0.002\ e^{-0.3(20-T)}$ ....(19) or equivalently substituting $\theta = e^C$

$$K_s = 0.002(1.35)^{-(20-T)} \ldots.(20)$$

The results from the numerical solution together with the experimental data are plotted in Figs. 20 and 21. The theoretical solution (Fig. 20) is determined from August 1963, when Matero North pond came into operation, until May 1965, but experimental results were only available from May 1964 to May 1965.

As ratios of gas evolution only are considered the absolute magnitude of the BOD settling as a sludge is not required. Consequently on the semilog plots in Figs. 20 and 21 the theoretical solution can be moved vertically up and down *en bloc* to check the mean fit. (This is equivalent to multiplying the gas evolution by various values corresponding to assumed BOD loads settling as sludge.

To verify the constants for the sludge layer, a solution was obtained for the "Oswald" pond in Fig. 21. Temperatures were only available for the surface of the pond liquid. As the temperature at the sludge layer is normally less than the day surface temperature, and if it is assumed that the day temperatures are the ones in fact reported by Oswald, the sludge temperature must be estimated from the surface temperature as follows: From equation (23) the relationship between the maximum daily surface temperature and the sludge temperature is given approximately by $T_s = 25/32T_T$. As it is unlikely that the temperatures reported by Oswald were maxima, the relationship $T_s = 27/32T_T$ was used. Assuming a sinosoidal yearly variation of temperature between the limits of temperatures reported, the theoretical ratios of gas evolution were calculated by solution of equation (11), together with the experimental data in Fig. 21. The slope of this plot is not greatly different from that given by the Matero pond data.

In the pond liquid the degradation rate can be determined from equation (10) with $\theta = 1.085$ and $K_{35} = 1.2$ at T = 35°C.

To incorporate the effect of the algae on the effluent BOD, the BOD in the pond is taken as the sum of two separate fractions - the BOD as a result of the degradation action [solution of equations (11) to (13)] and the BOD due to the algae. Considering the total BOD, in Fig. 22 a BOD-time curve for the pond effluent is shown obtained under the usual conditions laid down for this test. It is evident the curve is not of the first order type normally found in sewage processes. The algal respiration rate appears to be constant over the period of time of the test and tends to obscure the BOD demand of the bacteria. From this result and results of BOD tests on filtered and unfiltered pond effluents, it was concluded that the BOD, including the algae, (termed the BODAL), is approximately 3 times the BOD due to the pollution only, i.e. the algae absorbs about twice as much oxygen in 5 days as the bacterial oxidation process.

When the temperature in the pond liquid falls to lower levels, the reduction in the concentration of algae has the effect of reducing the multiplying constant for the algal BOD. At 15°C and above, the constant is assumed to be equal to 3, and at 0°C equal to one. Between these limiting values the multiplying constant is determined on the basis of a cosine curve reducing from one at 15°C to zero at 0°C, i.e.

$$BODAL_T = (BODAL_{15} - BOD_{Bacteria}) \text{ Cos } O + BOD_{bacteria} \qquad \ldots.(21)$$

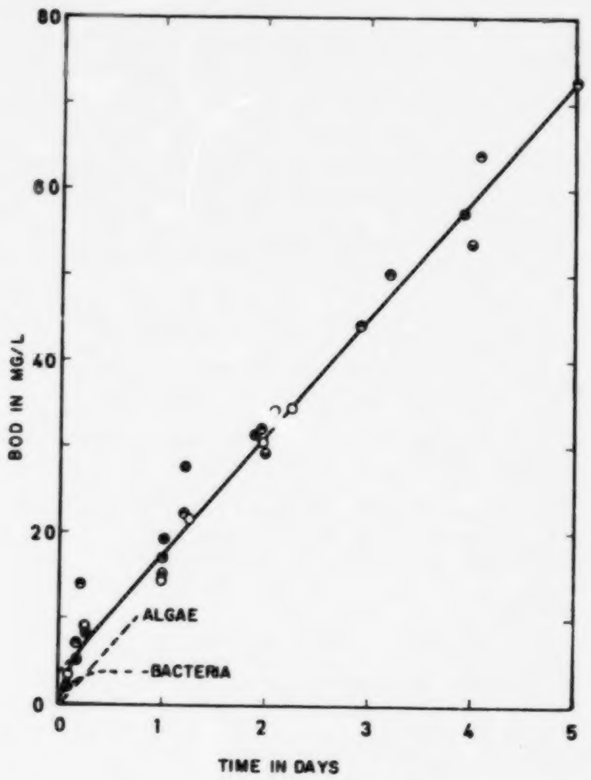


FIG. 22. BOD-time Curve for Primary Oxidation pond effluent.

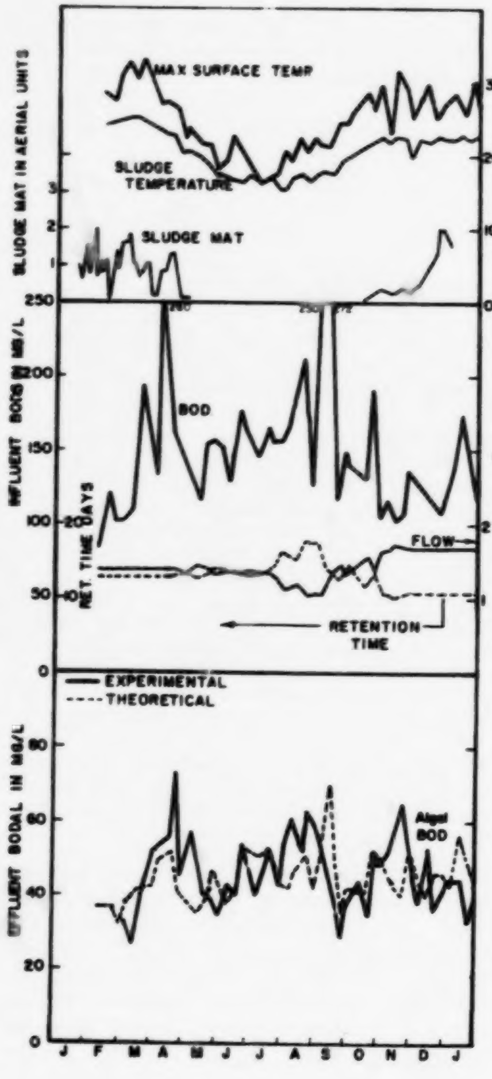


FIG. 23. Experimental and theoretical BOD's in Matero North Pond, Lusaka.

where $\cos\Theta = (15-T)\dfrac{\pi}{2 \times 15} \quad (0 \leqslant T \leqslant 15) \quad \ldots . (22)$

and Θ is in radians.

The temperature T in the pond is assumed to be the average between the maximum surface temperature during the day and the average daily sludge temperature.

EXPERIMENTAL VERIFICATION

As a test of the theory and the derived constants the numerial solution of equations (11) and (13) were obtained applied to Matero North pond. Data available were influent flow, volume of the pond, maximum daily surface temperatures and average daily sludge temperatures. Influent and pond BOD's were available at approximately weekly or two–weekly intervals. Numerical integration was performed using the time increments between sequential experimental BOD variables: these were on average 7 days but on occasion 14 days. The method of integration was that developed by Marais & Shaw[8], being the most accurate theoretically for this particular type of differential equation.

Calculated pond BODAL together with experimental values and other pertinent data are shown in Fig. 23.

As a further test of the theory under a very different set of environmental conditions, the solution of a pond in Canada was obtained. Data on this pond were kindly supplied by Mr. M.E. Czarnecki, of the Ontario Water Resources Commission, Toronto. In this pond lengthy periods of icing over occur. Furthermore the pond water level is lowered in the autumn in order to hold the winter flow. Under ice cover the pond turns anaerobic, but settlement of course continues to take place. It was assumed that when icing over occurs the degradation in the pond liquid occurred at the same rate as the anaerobic sludge layer.

Data was available on influent flows and influent and pond BOD's. Surface temperatures only were available, hence the sludge temperatures had to be estimated; this was done by using the relationship between the surface temperature, $T_s = 25/32 T_T$, for reasons describes before. It was assumed that the pond liquid volume was reduced to 20% at the beginning of November. Subsequently of course the pond volume increased with time as the influent flow accumulated until the pond overflowed. The computer program incorporated all these effects.

Solution of the pond BOD, together with the experimental and other pertinent data, are shown in Fig. 24. Correlation between the calculated and experimental BODAL values is good except for vaules near the end of the period. However, data on environmental conditions, such as periods of ice cover and the exact times of lowering the pond volume, were not available, which probably affects the results. The times of initiation and termination of ice cover are particularly critical as the pond BOD changes rapidly during the interval the pond is iced over.

When considering that the two ponds differed markedly in retention times (120 days versus 12 days) and surface temperature ranges (0-22°C versus 17-32°C) and that their theoretical solutions were determined using the same kinetic constants, the overall description given by the theory appears to give account to the observations.

DYNAMIC POND BEHAVIOUR AND DESIGN

Data for the design of ponds usually consist only of rough estimates of influent BOD and flow. Temperatures at the sludge layer and in the pond are generally not directly available although it has been demonstrated that these parameters are of great importance. Wind conditions can be approximated from meterological data. Any design method for general use must be based on the data that are available to the designer. Normally the designer is only interested in a simple method which will provide adequate performance under critical conditions. To arrive at the simple method may require extensive analysis of various assumed conditions; then by selecting critical points in the solution these can be plotted in the form of design charts. This is the approach that will be used in this section.

Meteorological data on air temperatures usually include three monthly average values: maximum–mean, mean and minimum–mean daily temperatures. Each of these has yearly maximum and minimum value. Taking, for example, the maximum–mean temperature, it will have a maximum mean–maximum and a minimum mean-maximum value. From Fig. 14 it is apparent that the mean–maximum temperature is approximately the same as the mean–maximum temperature of the pond surface layer. Hence an estimate of the average maximum pond surface temperature for each month of the year is available. In Fig. 25 the relationship between the maximum daily top surface temperature and the average daily sludge temperature for Matero North pond is shown. The relationship is linear and can be described by equation (23).

$$T_s = 0.777\ T_T + 0.89 \qquad \ldots .(23)$$

approximated to $T_s = 25/32\ T_T$

where T_s = sludge temperature in degC

T_T = maximum daily surface temperature in degC.

Note that in equation (23) the temperatures coincide at 4°C, which is reasonable for as temperatures decline below 4°C the density of the water decreases, and above 4°C, under falling temperature conditions, thermal mixing will occur and will tend to equalize the temperatures throughout the pond as it approaches 4°C. Below 4°C, further atmospheric cooling will stratify the liquid, the colder water remaining in the top layers, where it freezes. gradually downwards, the bottom waters tending to remain at 4°C. Thus the lowest temperature in the sludge can be accepted as 4°C, unless the whole mass freezes.

Usually the variation of air temperature over a yearly cycle approaches a sinusoidal slope. This will also apply to the mean–maximum temperature. If the maximum and minimum value of the mean–maximum temperature is known, the values in every month, or week, of the year can be determined by fitting a sine curve to pass through the maximum and minimum at 6–month spacings. In this fashion the maximum surface temperature and sludge temperature are known approximately and, consequently, the mean pond temperature are known approximately and, consequently, the mean pond temperature.

To determine the behaviour of a pond under different levels and yearly cyclic ranges of temperature a computer program was developed, reproduced in Appendix A. The input requires influent BOD5 flow, volume of pond, liquid depth, maximum and minimum mean–maximum air temperatures. The program assumes a sinusoidal variation of air temperature, calculates the temperatures in the sludge and in the pond, the various degradation rates and the

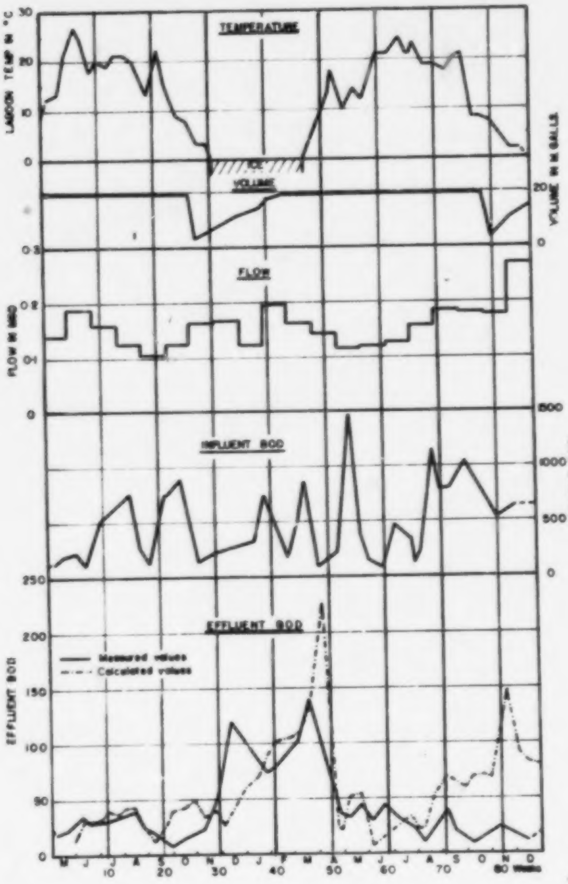
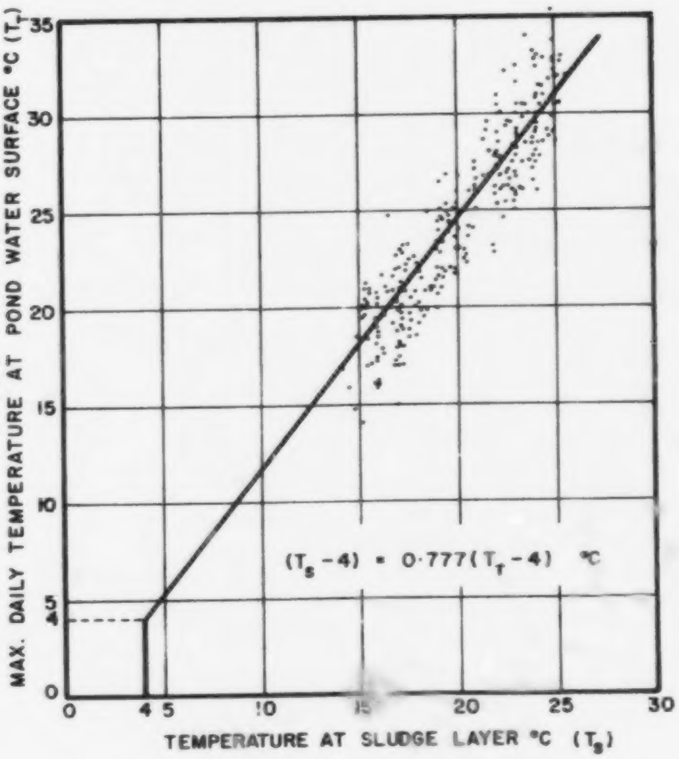


FIG. 24. Measured and calculated BOD effluent values for a pond in Ontario, Canada.

FIG. 25. Relationship between the maximum daily temperature at pond water surface and average temperature at sludge layer.



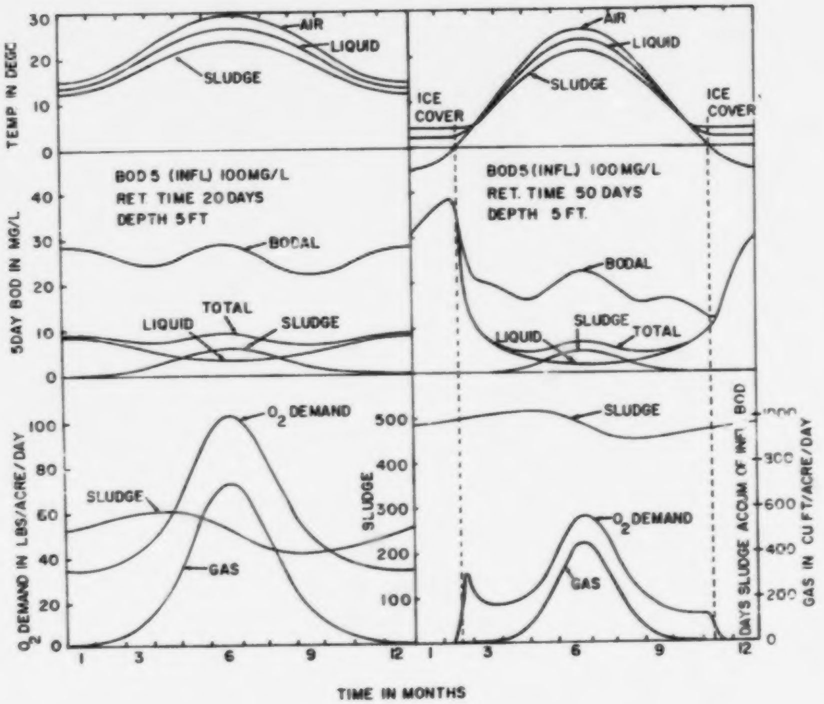


FIG. 26. Simulated behaviour of oxidation ponds.

BOD's in the pond and sludge, and the oxygen demand. Allowance is made for reduction of algal BOD due to temperature and for anaerobic action under ice cover. The program prints out, amoungst others, the various temperatures, degradation constants, bacterial pond BOD$_5$'s due to the influent fraction direct to the pond and the feedback from the sludge, the total BOD$_5$ including the algal and the oxygen consumption rates due to the bacterial action. Any number of years of information can be developed. Detailed account of the program is tedious, but the listing in Appendix A supplies sufficient explanatory comments to be clear to a person versed in programming.

In Fig. 26 the variation of the different pond parameters is shown for two ponds with temperature ranges 15–30°C and -5-25°C respectively. The influent BOD is equal to 100 mg/l and retention time equal to 20 and 50 days respectively. The plot depicts the situation after 20 years' operation when the sludge mass is in virtual dynamic equilibrium. In Fig. 27 the change in the sludge mass with time over the first 6 years' operation is shown.

Interesting points to note are:

1. The BODAL in the 15 to 30°C pond hardly varies over the year's cycle despite the large individual changes in the in the BOD's due to the sludge feedback and the non–settleable influent fraction. In the -5 to 25°C pond the BOD maximum occurs in the winter months, the BOD increasing rapidly under ice cover, but falling equally rapidly once aerobic conditions are re–established in the spring.
2. The oxygen demand due to bacterial degradation changes appriciably between winter and summer even though, as in he 15–30°C pond, the pond BOD5 varies little. Consequently the pond BOD is not a sufficient parameter by which the onset of anaerobic conditions can be ascertained.
3. The sludge mass increases, but at a decreasing rate, from year to year eventually to reach equilibrium after a number of years depending upon the average level of temperature and the temperature range over the yearly cycle. The feedback from the sludge to the pond concomitantly increases, consequently the pond does not deal with the full effect of the influent BOD load until several years have passed. *This serves as a warning against the practice of testing performance of ponds over short periods after the inauguration of, or, after a change of, influent BOD load to the pond.*

A chart which assists in the design of ponds under the various cyclic temperature changes can be developed as follows: Using a constant retention time of, say, 20 days and a constant influent BOD$_5$ of, say, 100 mg/l the effluent BOD (BODAL) is calculated by means of the computer program for a number of ranges between the maximum mean–maximum and minimum mean–maximum temperatures. The calculation in each case is continued until equilibrium cycle conditions are attained. The maximum BODAL pond value in the year's cycle is selected and substituted in equation (3) written as follows:

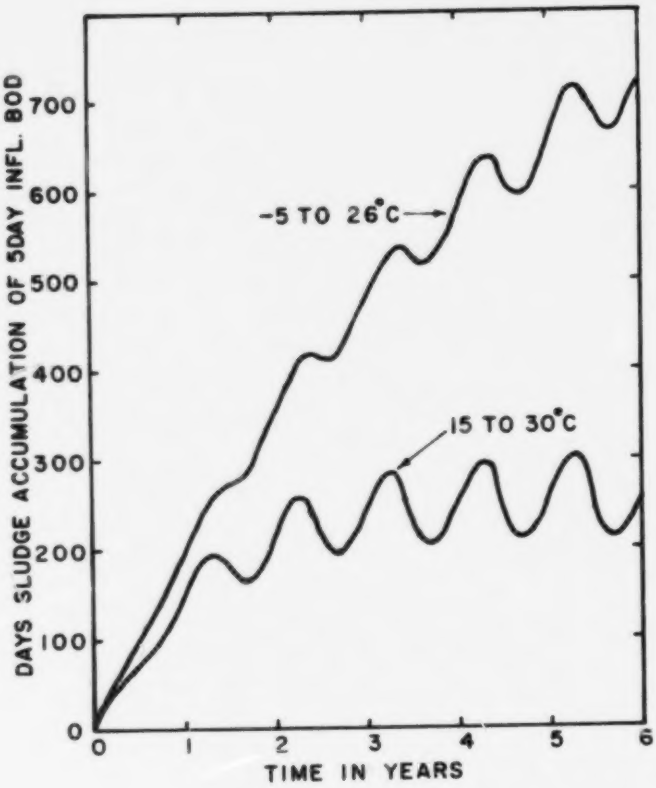


FIG. 27. Sludge accumulation in oxidation pond under different annual temperature cycles.

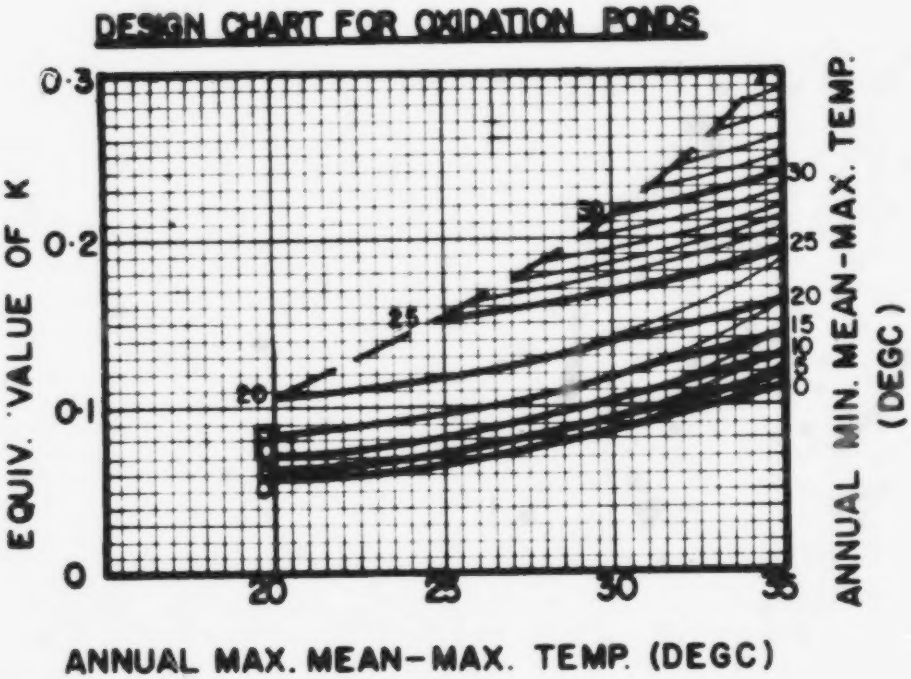


FIG. 28.

$$K = (P_o/P-1)/R$$

Where

P = maximum BODAL 5-day value in mg/l

P_o = influent BOD5 in mg/l and solved for K to give equivalent K value, K_e.

From a number of these calculations the chart shown in Fig. 28 is drawn to give the value of K_e for any combination of maximum and minimum mean–maximum air temperatures. These values of K are independent of the retention time, R: this can be checked by repeating the calculations for, say, R = 40 days. This is the only time when the pond is in dynamic equilibrium.

Assume for a moment the maximum effluent BODAL from the first pond in a series must not exceed 60 mg/l (in order to reduce the probability of anaerobism), design a pond for an influent BOD 300 mg/l, maximum mean–maximum temperature 32°C, minimum mean–maximum temperature 10°C. From Fig. 28, K_e = 0.11, hence R = (300/60–1)0.11 =37 days.

The BOD in the pond due to degradation action will be approximately one–third of the pond BODAL value, i.e. 13 mg/l. If discharged to a second pond it is unlikely that the effluent will decrease below (37–13) = 26 mg/l, as the algae is not significantly reduced in subsequent ponds. The faecal bacteria will, of course, be further reduced.

It should be noted that the design chart does not incorporate values of the minimum mean–maximum temperature below 0°C. Where ice cover is encountered, the effluent quality is very sensitive to the time–duration of cover; furthermore, the pond is then inevitably anaerobic and a different criterion governs the design. In North America, for example, the period of recovery from anaerobic conditions to aerobic conditions during the Spring thaw is accepted as a criterion, for during this period odours are given off by the exposed anaerobic liquid. Consequently the ponds are given long retention times so that the pond BOD increase is moderated during ice cover and aerobic conditions are likely to be rapidly re–established during the Spring thaw. Of course, once the pond size has been determined it will be possible to make an estimate of the BOD variation in the pond over a year from the computer program listed in the appendix. Generally, the sizing of ponds in the cold–temperate regions must be based on the past experience re–development of odours during the thaw.

In warm–temperate and tropical regions ice cover does not occur. However one still has the problem of deciding to what level the pond may be loaded before anaerobic conditions will become dominant. Marais & Shaw [8], from a summary on pond BOD data, proposed that the maximum pond BOD, P_m should not exceed

$$P_m \ngtr 750/(0.6d+8)$$

where d = depth of pond in feet.

The constant 750 was later reduced 600 by Meiring, Drews, van Eck & Stander[13] which in a 5ft deep pond reduces the maximum allowable pond BOD from 68 to 55 mg/l. This leads to a considerable increase in the pond area, particularly if the determination is coupled with the generally lower values of K_e proposed in this paper compared to that advocated by Marais & Shaw, viz. 0.17. It is suggested that, with the present lack of knowledge on the advent of anaerobic conditions in the pond, the constant should be approximately 700.

In this paper it has been demonstrated that the wind velocity has a pronounced effect on algal growth and re–oxygenation capacity of the pond and, consequently, on the development of anerobic conditions. Where possible it is suggested that the wind velocities should be plotted in Fig. 8, and an indication obtained as to the month in which lack of mixing in the pond may be critical for anaerobic conditions. Not only should the average wind conditions be considered, but also the behaviour pattern of the wind during the day. In Cape Town, for instance, the wind is very low during the morning and rises to a maximum at approximately 4 o'clock in the afternoon. Under such established, the stratification can withstand quite high velocities of wind without being destroyed. The wind mixing effect in therefore uncertain. In Zambia, the windy part of the day occurs early in the morning, so that mising normally occurs every day except under very low early wind velocities.

From the data reported in this paper, the favourable influence of mixing is so pronounced that the writer is convinced that there is a place in oxidation pond design for inducing artificial mixing. In order to mix the maximum quantity of water most efficiently, axial propellor type pumps should be installed. The capacity of the pump should be sufficient to give a turnover of once every 12 hours, preferably during the day. The discharge velocity should be in the vicinity of 3ft per second, which will give sufficient momentum to the discharge to stream away from the pump and assist in the mixing action. Tests are to be inaugurated at the City of Cape Town's oxidation ponds into the effect of artificial mixing. Data obtained from these experiments should increase our knowledge of the mixing action on the behaviour of ponds.

Alternative methods of improving the loading capacity of ponds are by anaerobic pretreatment. If the critical conditions develop in summer, the pond volume can be greatly reduced by anaerobic pretreatment of the effluent in tanks or in open ponds. These facilities reduce the fermentation potential of the sludge layer in the primary pond and practically eliminate rising sludge during the hot season.

In small communities of up to 10,000 people, treatment in tanks is preferred for once alkaline anerobic fermentation is established odour development is virtually non–existent. The method has been widely applied in Zambia with great success. Retention times in the tanks range from about 3/4 to one day. BOD reduction in the tank can be taken as of the order of 40–50% for design purposes. The design of oxidation ponds receiving pretreatment effluent is exactly the same as for raw sewage except that the effluent BOD from the pretreatment facility is used in the design. Details of this system will be discussed in a future paper.

In large facilities it may be preferable to use anaerobic lagoons open to the atmosphere. The problem of odours is an ever present one, but from the work on the ponds at Durban[6][14] it appears that the recycling from the first or second oxidation pond to the influent of the anaerobic pond in the ratio of 1 : 6(recycle flow : raw sewage flow) very drastically reduces, or completely eliminates, odour development. The reason for this is not yet clear, but the effect is conclusively established and should be utilized where supervision and power are available. Anaerobic pretreatment of effluent is so advantageous that the first consideration in the design of a series of oxidation ponds should always include the possibility of anaerobic pretreatment.

CLOSURE

A model which reasonable describes the observed behaviour of a phenomenum often leads to a general reassessment of the relative importance of the different parameters used in its construction. It may focus attention on parameters previously neglected and may indicate that some parameters thought important are, in fact, not so. In this fashion it provides a framework for future experimental and theoretical investigations. For example, it is clear from the model presented in this paper that: (1) mixing in the pond demands greater attention in both theoretical and practical aspects; (2) advent of anaerobic conditions in aerobic oxidation ponds is a more complex phenomenum than previously thought and is apparently dependent upon temperature, mixing conditions, algal growth and BOD in the pond and, possibly, other parameters; (3) the sludge layer occupies an important part in the degradation process and directs attention to the benefits in partial separation by anaerobic pretreatment.

The model described in this paper will probably undergo great changes with time. It is unimportant whether the model gives only a partial description of the phenomenum provided it is fertile in inducing new ideas which may lead to improvement in this important method of sewage purification.

ACKNOWLEDGEMENTS

This research would have died in infancy had it not been for Mr. L.J. Vincent, Manager, Zambia Housing Board, whose enthusiasm inspired all those who worked under him. His concept on the needs and the future of low cost sanitation greatly influenced the author and gave impetus to this research. Acknowledgement is due to the Board under whose wing this research was done, who provided the finance and gave permission for the data to be used in this report. The author was ably assisted by Mr. P. Berry-Otteway, Research Assistant to the Board, who did most of the experimental work.

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APPENDIX A
SIMULATION OF OXIDATION PONDS.

FOR
ONE WORD INTEGERS
LIST SOURCE PROGRAM
IOCS (CARD,TYPEWRITER,KEYBOARD,1132PRINTER, DISK,PLOTTER)
ARITHMETIC TRACE
TRANSFER TRACE
IBM 1130 BASIC FORTRAN IV

SIMULATION OF PERFORMANCE OF FACULTATIVE PONDS
PROGRAM BY KUECK AND MARAIS
CIVIL ENGINEERING DEPARTMENT
UNIVERSITY OF CAPE TOWN
RONDEBOSCH, SOUTH AFRICA

THE HERMANN + GLOYNA AND MARAIS + SHAW THEORIES OF THE DEGRADATION ACTION IN OXIDATION PONDS HAVE BEEN INTEGRATED, AND ACCOUNT IS TAKEN OF THE EFFECT OF THE SLUDGE LAYER.

THE OPERATION OF THE POND IS CALCULATED FOR INTERVALS OF ONE WEEK, REPEATEDLY FOR A GIVEN PERIOD OF YEARS.

REFERENCE --
BULL. WLD HLTH ORG.
1966. 34, 737-763
G. V. R. MARAIS

INPUT PARAMETERS
NOTE – EACH JOB REQUIRES 6 CARDS OF INPUT DATA.
(SEE READ STATEMENTS)
ANY NUMBER OF JOBS MAY BE RUN IN SUCCESSION.
LAST CARD OF LAST JOB MUST BE FOLLOWED BY A BLANK CARD

NAME = DESCRIPTIVE IDENTIFICATION OF JOB

KSO,KPO = DEGRADATION CONSTANTS FOR SLUDGE AND POND LIQUID

THETE,THETA = ARRHENIUS CONSTANTS FOR SLUDGE AND POND LIQUID

IP = FRACTION OF INFLUENT BOD TO POND LIQUID

SP = FRACTION OF FERMENTATION PRODUCTS FROM SLUDGE LAYER TO POND LIQUID

SP FRACTION OF FERMENTATION PRODUCTS FROM SLUDGE LAYER TO POND LIQUID

Q = FLOW (MILL. IMP. G.P.D.)

DEPTH = OF POND (FEET)

RET = RETENTION (DAYS)

BOD I5 = INFLUENT 5–DAY BOD (PPM OR MG/L)

TMIN = ANNUAL MINIMUM OF MEAN DAILY MAXIMUM AIR TEMPERATURE

TMAX = ANNUAL MAXIMUM OF MEAN DAILY MAXIMUM AIR TEMPERATURE (ALL TEMPERATURES ARE GIVEN IN DEGREES CENTIGRADE)

NTMAX = WEEK END OF HIGHEST TEMP., COUNTED FROM 1ST JAN.

NS = WEEK IN WHICH OPERATION COMMENCES, COUNTED FROM 1ST JAN.

NNSTA = YEAR IN WHICH PRINT–OUT IS TO COMMENCE

NNSTO = YEAR IN WHICH OPERATION AND PRINT–OUT ARE TERMINATED

NOTE - DEGRADATION CALCULATIONS ARE REPEATED FOR INTERVALS OF ONE WEEK' COMMENCING AT WEEK 'NS' OF THE FIRST YEAR. RESULTS ARE PRINTED–OUT FORTNIGHTLY, COMMENCING AT WEEK 'NS' IF NNSTA = 1, OTHERWISE COMMENCING AT THE BEGINNING OF YEAR 'NNSTA'. CALCULATION AND PRINT–OUT ARE TERMINATED AT THE END OF YEAR 'NNSTO'.

INTERNAL VARIABLES

PUI = INFLUENT ULTIMATE BOD (P.P.M.)

AREA = OF POND (ACRES)

V = VOLUME OF POND (MILL. IMP. GALLS)

TDIF = HALF OF RANGE OF TEMPERATURE

TSUM = AVERAGE ANNUAL TEMPERATURE

N = WEEK COUNT

(N) = SUBSCRIPT REFERING TO BEGINNING OF WEEK 'N'

(N+1) = SUBSCRIPT REFERING TO END OF WEEK 'N'

TA,

TEMPA = AIR TEMPERATURE

TS,

TEMPS = TEMPERATURE OF SLUDGE

TP,

TEMPP = AVERAGE TEMPERATURE OF POND LIQUID

KS,KP = DEGRADATION RATES FOR SLUDGE AND POND LIQUID

BODAY = WEIGHT OF INFLUENT ULTIMATE BOD LOAD (LBS/DAY)

PUP = ULT. BOD IN POND DUE TO NON–SETTLING INFLUENT

BODP5 = 5–DAY IN POND DUE TO NON–SETTLING INFLUENT

PUS = ULT. BOD IN POND DUE TO SLUDGE FERMENTATION

BODS5 = 5–DAY BOD IN POND DUE TO SLUDGE FERMENTATION

PU = TOTAL ULT. BOD IN POND (MG/L)

BOD5 = POND %-DAY BOD

S = TOTAL WEIGHT OF SLUDGE IN POND (LBS.)

DLSDG = TOTAL SLUDGE' AS DAYS OF AVERAGE INFLUENT ULT. BOD LOAD

NN = YEAR COUNT

BODAL = POND 5–DAY BOD INCLUDING ALGAE

BOD5I = WEIGHT OF INFLUENT 5–DAY BOD LOAD TOTAL (LBS/DAY)

BODPA = WEIGHT OF INFLUENT 5–DAY BOD LOAD PER ACRE

D = RECIPROCAL OF RETENTION TIME

O2, = OXYGEN DEMAND (LBS/ACRE/DAY)

OPHR = OXYGEN/HOUR (MG/L,PPM)

SPA,

ISPA = WEIGHT OF SLUDGE IN POND (LBS/ACRE)

GAS = TOTAL GAS PRODUCED (CU.FT./DAY)

GASPA = GAS PRODUCED (CU.F/ACRE/DAY)

GASLB = GAS PRODUCED (CUFT./DAY/LB. OF ACCUMULATED SLUDGE)

```
REAL KSO,KS (53),KPO,KP (53),IP
DIMENSION NAME (80) , TA(53), TS(53), TP(53)
70 WRITE (3.120)
READ (2,109) NAME
READ (2,110) KSO, THETE, KPO,THETA
READ (2,111) IP,SP
READ (2,112) Q,DEPTH,RET,BOD15
READ (2,113) TMIN,TMAX
READ (2,114) NTMAX,NS,NNSTA,NNSTO
WRITE (3,99) NAME
WRITE (3,100) KSO,THETE,KPO,THETA
WRITE (3,101) IP,SP
WRITE (3,102) Q,DEPTH,RET,BOD15
WRITE (3,103) TMIN,TMAX
WRITE (3,104) NTMAX,NS,NNSTA,NNSTO
FP=IP
CP=SP
FS=1.0–FP
PUI=BOD15*1.47
AREA=(Q*1000000.*RET)/(6.25*DEPTH*43560.)
V=Q*RET
TDIF=(TMAX–TMIN)*0.5
TSUM=(TMAX+TMIN)*0.5

SETTING–UP WEEKLY DEGRADATION RATES

XINCE=3.141593*2.0/52.0
DO 10 N=1.53
XN=N-1
TA(N)=TSUM+TDIF*COS((XN–NTMAX)*XINCR
IF (TA(N)) 7,7,8
7 TS (N)=4.0
TP(N)=2.0
KS(N)=KSO*THETE**(TS(N)–20)
KP(N)=KS(N)
GO TO 10
8 IF(TA(N)–4.0)12,12,11
```

```
12 TS(N)=4.0
GO TO 13
11 TS(N)=0.777*TA(N) + 0.9
13TP(N) =(TA(N)+TS(N))*0.5
KS(N)=KSO*THETE**(TS(N)-20)
KP(N)=KPO*THETA**(TP(N)-35)
10 CONTINUE
BODAY=Q*PUI*10.
BOD5I=BODAY/1.47
BODPA = BOD5I/AREA
D=1.0/RET
PUS=0.0
PUP=0.0
S=0.0
DSLDG=0.0
     WRITE (3,54) AREA,BOD5I,BODPA
NN=1
43 IF(NN−NNSTA) 39,38,38
38 WRITE (3,120)
IF (NN−3) 40,41,46
40 IF(NN-1) 44,44,45
41 WRITE (3,53)
GO TO 47
44 WRITE (3,55)
GO TO 47
45 WRITE (3,56)
GO TO 47
46 WRITE (3,57) NN
47 WRITE (3,59)

WEEKLY ACCUMULATION

39 DO 34 N=NS,52

SOLN OF DIFF EQUNS REFERENCE MARAIS,
G.V.R, AND SHAW, V.A. (1961) TRANS. S. AFR.
INSTN. OF CIVIL ENGRS, 3,250

NOTE -- TIME INCR.,D, IS CONST. =7
SLUDGE ACCUMMALATION
SS=S
S=(S*(2.-7.*KS(N)) +2.*7.*(FS*BODAY))
                              /(2. + KS(N+1)*7.)
EFFECT OF SLUDGE FERM. ON ULT. POND BOD
PUS=(PUS*  (2-*7 *(KP(N)  +D))  +
CP*7*(KS(N)*SS+KS(N+1)*S)/(V*10))/
1(2+7*(KP(N+1 +D))
EFFECT OF NON-SETTL. INFLUENT ON ULT.
POND BOD
PUP=(PUP*(2-7*(KP(N)+D))+2*7*FP*PUI*D)
                              /(2+7*(KP(N+1)+D))

SUM OF EFFECTS
PU=PUS+PUP IF(NN-NNSTA) 34,42,42

PRINTING-OUT AT FORTNIGHTLY INTERVALS

42 IF ((N/2)*2-N) 34,35,35,
CONV. TO 5-DAY BOD BY MULT. BY 1/ .3 AS
SOLID
SETTLED BOD IS NOT PRESENT
35 BOD5=PU/1.3
BODS5=PUS/1.3
```

```
BODP5=PUP/1.3
IF (TA(N+1)) 60,60,61
60 BODAL=BOD5
GO TO 62
CALC. AN EQUIV. K VAL.
61 EK=(BOD15/BOD5−1)*D
DET. EFFECT OF ALGAE ON BOD BY REDUCING
K VALUE EK=EK/4
DET. BODAL USING EK
BODAL=BOD15/(EK*RET+1)  THIS  METHOD
    ENSURESLINEARITY OFBODAL/BOD5 FOR
    ALL RETENTION TIMES
ALTERNATIVLY----MULT.BOD5 BY 3 WHICH
GIVES VERY
    NEARLY THE SAME VALUE BOT NOW
    BOD15/BOD5 FOR ANY
    POINT ON YEAR CYCLE ONLY APPROX.
    LINEAR
TEMP. EFFECT ON ALGAE
IF (TP(N+1)−15.) 63,62,62
63 TTH=(15-TP(N+1))*3.1416/30
TTH=COS(TTH)
BODAL=(BODAL−BOD5)*TTH+BOD5
DAYS SLUDGE ACCUM. IN TERMS OF DAILY
INFLNT BOD
62 DSLDG=S/BODAY
OXYGEN/ACRE/DAY (LB)
02=PU*KP(N+1)*V*10/AREA
OXYGEN/HOUR (MG/L) EXCLUDING ALGAL
RESPIRATION
OPHR=PU*KP (N+1)/24.0
DAYS SLUDGE ACCUM./ACRE
SPA=S/AREA
GAS=S*10*KS(N+1)*(1-CP)/1.47
GASPA=GAS/AREA
GASLB=GAS/S
EQUIV.VALUE FOR K (INCLUDING ALGAE)
DK=(BODAL−1)*D
TEMPS=TS(N+1)+0,05
TEMPP=TP(N+1)+0.05
IF(TA(N+1)) 30,31,31
30 TEMPA=TA(N+1)−0.05
GO TO 28
31 TEMPA=TA(N+1)+0.05
28  WRITE(3,52)  N,TEMP,TEMPS,TEMPP,KS(N+1),
KP  (N+1),DSLDG,KP,PU,1BODS5,BODP5,BODAL,
C2,OPHR,GASPA,GASLB
52  FORMAT(I4,3F6.1,F9.6,F8.1,F7.4,F8.2,4F7.2
F7.1,F7,3,  1F8.1,F8.4)
34 CONTINUE
NS=1
NN=NN+1
IF(NN−NNSTO)43,43,70
53  FORMAT(5X,'THIRD YEAR OF OPERATION')
54  FORMAT (//' AREA INFLUENT 5-DAY BOD
LOAD PER DAY'/' 1 ACRES TOTAL (LBS/DAY)
 LBS/ACRE/DAY'/1X,F14.4,6
2X,F12,2,5XF12.2)
55  FORMAT (5X,'FIRST YEAR OF OPERATION')
```

```
56 FORMAT (5X,'SECOND YEAR OF
OPERATION')
57 FORMAT (5X,13,'TH YEAR OF OPERATION')
59 FORMAT(' TEMPERATURES DEGRADATION
ACCUM EQUIV
1----BODS IN POND (PPM) --- 02 DEMAND
GAS PRODUCED '/
2' WEEK O C RATES SLUDGE K ULTIMATE
3----5-Day---- LBS/DAY MG/L CUFT/DAY
PER '/
4' -END AIR SLDG LQD SLUDGE LIQUID DAYS
DAYS-1 TOTAL
5SLDG + LIQD =TOTAL ALGAL /ACRE /HOUR
ACRE LB SLDG')
99 FORMAT (///10X,BOA1///' INPUT---'//)
100 FORMAT (' KSO THETE KPO THETA '4F10.3)
101 FORMAT (/5X,'FRACTIONS TO POND LIQUID
--- FROM INFLUENT =',F6.3,',
1 FROM SLUDGE =',F6.3)
102 FORMAT (/' FLOW DEPTH RETENTION
BOD15 '/' MGD
1 FT DAYS PPM '/4F10.3 )
103 FORMAT (/' TMIN TMAX '/2F10.3,' DEG.
CENT.')
104 FORMAT (/5X'HIGHEST TEMP OCCURS AT
END OF',13,'TH WEEK'/5X,'OPERA
1TION COMMENCES IN', 13,' TH WEEK OF FIRST
YEAR'/5X,'FORTNIGHTLY PRI
2NT-OUT COMMENCES IN',13,'TH
YEAR'/5X,'OPERATION REPEATED UP TO END
3 OF',13,'TH YEAR'///' OUTPUT ---')
109 FORMAT (80A1)
110 FORMAT (4F10.3)
111 FORMAT (2F10.3)
112 FORMAT (4F10.3)
113 FORMAT (2F10.3)
114 FORMAT (16.3110)
120 FORMAT (1H1)
CALL EXIT
END
```

```
XEQ
 POND DESIGN
   0.002        1.350     1.200     1.085
   0.400        0.400
   1.000        5.000     20.000    100.0
   15.000       30.000
   26           1         1         20

 POND DESIGN
   0.002        1.350     1.2000    1.085
   0.400        0.400
   1.000        5.000     50.000    100.0
   -5.000       26.000
   26           1         1         20
```

DEVELOPMENT OF LAGOON DESIGN STANDARDS IN UTAH

by
Calvin K. Sudweeks, Executive Secretary
Utah Water Pollution Committee*

BACKGROUND

In many areas of Utah limited stream flow minimizes potential for dilution of treated effluents. In other areas, receiving waters are virtually nonexistent and effluent quality is literally the quality of the downstream flow. As a result, the need for high quality wastewater treatment was recognized early and secondary treatment with chlorination was established as a basic requirement following passage of the Utah Water Pollution Control Act in 1953.

This Act created a Water Pollution Control Board (9 members) within the framework of the Utah State Department of Health. The Board was specifically charged with the conduct of an effective water pollution control program. It was required to place first in priority those control measures which would eliminate pollution creating a hazard to public health; to develop and assign water quality standards; to abate existing pollution; and to control new sources of pollution.

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*Formerly Utah Water Pollution Control Board

Among the early Board accomplishments was establishment of water quality standards, with designations "A" through "E". By Utah's definition, Class "A" and "B" waters are essentially ground waters obtained from wells and springs.

Class "C" relates to surface waters suitable, without treatment, for irrigation, stock watering, propagation of fish and wildlife, recreation (except swimming), industrial water supplies, and for domestic water supplies when subjected to complete water treatment processes. Biochemical oxygen demand (BOD) and MPN coliform limits for Class "C" waters are 5 milligrams per liter (mg/l) and 5,000 per 100 milliliters (ml) respectively.

Class "D" also relates to surface waters which are suitable for certain restricted uses, including limited irrigation and certain industrial supplies. BOD_5 and MPN coliform limits for the Class "D" waters are 25 mg/l and 5,000 per 100 ml respectively.

Class "E" waters are essentially useless and must be suitably isolated within conduits or fenced areas and otherwise controlled to minimize health hazards. Raw or partially treated domestic wastewaters are examples of Class "E" waters. The standards have recently been revised to include other classifications which will not be discussed here.

Another early action of the Board was the adoption of design standards, the original version being very similar to the so-called "Ten State Standards". These standards emphasized such secondary treatment facilities as trickling filters and activated sludge units (incorporating effluent disinfection) the design principles for which were sufficiently well established that effluent quality was reasonable predictable. In view of the recognized need for high quality secondary treatment (in many cases effluent consistently meeting Class"D" quality standards), the desirability for standard mechanical type treatment plants was obvious.

CONSIDERATION OF LAGOONS

In the latter 1950's, raw sewage lagoons were gaining increased prominence as a treatment device, and, with the appearance of popular articles of questionable scientific substantiation, many questions were posed regarding the possible use of such facilities in Utah. Although not then acceptable in Utah, it was well recognized that many states were utilizing raw sewage lagoons as waste treatment devices, and because of the growing interest an intensive review of available information was initiated in an attempt to determine the extent of available pertinent knowledge and whether indeed such facilities could be designed with confidence suitable for our needs.

In early considerations an attempt was made to find answers to many questions including the following:

1. Are lagoon design criteria sufficiently reliable that effluent quality will be consistently predictable with respect to such parameters as BOD and MPN coliforms; and are design parameters sufficiently clear cut to permit such design?
2. Does a serious problem exist with respect to distribution of infectious pond agents by wind, waterfowl and insects?
3. Are there any adverse effects on waterfowl?
4. Will it be practical to keep birds off ponds, such as is possible in mechanical plants?
5. Is it possible to design ponds so as to effectively reduce the algae population in the final effluent?
6. In the event effluent chlorination is necessary for coliform control, what usage of chlorine would be expected?

As the search continued it became clearly apparent that definitive answers to many of these questions were not available, including the question regarding design for a predictable effluent quality. In spite of these difficulties pressures for acceptance of lagoons continued to mount from all levels. In consequence thereof and by popular request, the Water Pollution Control Board in 1961 held a public discussion to explain its position on lagoons and to hear pros and cons of all interested persons.

Shortly before this time, the Utah Attorney General had issued an opinion holding that the Water Pollution Control Board was without jurisdiction in the case of lagoons which had no overflow. Thus it became possible for anyone to build such a lagoon without sanction of the Board providing, of course, P.L. 660 funds were not requested.

In view of this circumstance, the Board had lost any authority it may have had previously to prevent the hazard of exposed raw sewage in the environment; or, to put it another way, any hazards existing from such exposure were now defined as acceptable under Utah law.

The U.S. Public Health Service had earlier offered technical consultation on the problem of raw sewage lagoons and this offer was gratefully accepted at a meeting held July 26, 1963. Expert consultants from both the Public Health Service and from one of the Western States attended this meeting, at which time a full review of available information, including operating data, was covered.

A review of the raw sewage lagoon situation in the State represented revealed that such facilities were actively promoted as completely acceptable sewage treatment devices. That State preferred designs on the basis of BOD loading (20 lbs. per day per acre) and attempted to attain 100 to 120 day detention of flow because of the difficulties imposed by the winter season. Two-cell installations were reportedly preferred because they produced a better quality of effluent.

It was learned that MPN coliform measurements were seldom made on lagoon effluent in that State, and acceptable functioning rested principally on the basis of BOD removals and freedom from odors. It was comfirmed that BOD analyses are complicated by the presence of algae to the extent that comparison of results from different areas is difficult. It was further learned that a special study of coliform removals by lagoons in that State apparently indicated good reductions percentage-wise, but final effluents generally had counts substantially in excess of Utah's requirements for water of acceptable quality.

As reported by Public Health Service representatives at that time, unpublished data from brief studies at an eastern location showed lagoon effluent MPN coliforms ranging from 185,000 to 350,000 per 100 ml. It was also their feeling that a reasonably clear lagoon effluent could be chlorinated with good results. Further, it was agreed by all of the consultants that raw sewage lagoons in the environment would not constitute an undue risk to the public health—if properly sealed, fenced and isolated. "Kid-proof" fencing was thought to be desirable, though some states did not require it. The opinion was expressed that wind-blown contamination was of greater hazard from trickling filters and activated sludge plants than from raw sewage lagoons, and that the waterfowl hazard was unimportant, notwithstanding recognition of the fact that waterfowl can be excluded more readily from mechanical treatment plants than from lagoons. Public Health Service consultants stressed the importance of sealing, fencing and isolating lagoons, and pointed out that Utah's lack of jurisdiction over non-overflow lagoons represented a serious deficiency in these respects. It was felt that improperly sealed lagoons represented an unacceptable risk in areas of groundwater use.

The consultants agreed that Utah's water quality standards were not unreasonable, and further, that chlorination of sewage treatment plant effluents and lagoon effluents would be necessary when receiving waters are used recreationally, or for irrigation, or in those instances where substantial dilution is lacking, or where it was necessary to meet an accepted standard.

Other general conclusions developed in the discussion included the following:

Soil tests should be made in connection with all lagoon construction to determine sealing needs.

Lagoons should be operated at depths between 3 ft. and 5 ft. with an absolute minimum of 2 ft. permitted under special circumstances.

High sulfates in the community water supply may result in lagoon odors, but in one case in another state this did not prove to be a problem.

Series operation of lagoon cells will enhance the quality of effluent if enough cells are used.

Storage of the entire incoming flow is desirable during extreme winter weather.

Lagoon effluents will require higher chlorine dosages for satisfactory disinfection than will mechincal plant effluent, but total use of chlorine may be about the same due to reduced volume of lagoon effluent occasioned by evaporation and seepage.

Chlorine contact requirements likely would be about the same for both types of effluent.

Isolation of lagoons from human habitation should be at least 1,000 feet.

Lagoon dikes should have a minimum surface width of 8 ft. to facilitate vehicular access.

Reserve area equal to that of the initial lagoon should be set aside for future expansion.

DEVELOPMENT OF STANDARDS

From these discussions and continuing review of the literature it became increasingly evident that the generally accepted surface loading concept was based principally on control of odors and that this parameter did not relate in a precise manner to quality of final effluent. It also became evident that inlet considerations and pond configuration requirements were aimed at accomplishing fairly rapid dispersion of incoming flows throughout the pond. This unfortunately appeared to increase the short-circuiting tendency, a condition obviously contributing to deterioration of effluent quality, particularly in terms of coliform density.

It was further found that results reported from series-operated multiple ponds demonstrated rather dramatic improvements in final effluent quality. These improvements appeared more closely related to detention than to other factors, particularly as far as bacterial quality was concerned.

As the Board and staff continued to wrestle with the problem, there was a gradual withdrawal from the area loading concept in favor of the philosophy that detention time was a prime factor in designing for a specific end result of treatment.

Finally, it was decided that area loading requirements for the primary pond, where most odor problems are likely to develop, should be changed to a recommendation rather than a requirement and that the basic design specifications should be tied to overall detention period, combined with a requirement for a minimum number of ponds operated in series.

By this time it had also been concluded that final effluent disposal conditions in Utah justified adoption of certain effluent standards related directly to health protection. As a consequence, the design of all treatment facilities, including lagoons, was to be aimed at production of a final effluent having no more than 5,000 coliforms per 100 ml.

To arrive at specified design guides the Board and staff needed answers to many questions, including the following:

1. What detention period will assure final coliform counts below 5,000 per 100 ml?
2. What allowances should be made for seasonal treatment variations?
3. What factors are necessary to assure effective chlorination of lagoon effluent for coliform control?
4. How many ponds are necessary to provide reasonable assurance that short-circuiting is minimized sufficiently to produce consistent effluent results?

Unfortunately, clear cut answers to these questions were still not available, thus requiring that the Board and staff utilize the limited scientific information that was available to form the basis of general design decisions.

For example, the literature indicated that detention periods as long as 9 months might fail to guarantee coliform counts below 500,000 per 100 ml under winter conditions in two-celled ponds and that BOD levels might be far above 50 mg/l. The literature also indicated that 6 to 7 ponds in series might produce coliform counts around 5,000 per 100 ml with detention periods of as little as 1 month under semi-tropical conditions. Effluent BOD levels under 50 mg/l appeared feasible with 6 or 7 ponds operating in series, in moderate climates, and with detention periods of about one month. With respect to the question of algae control, no specific answer could be found, but dramatic reductions were shown to occur in instances of multiple ponds operated in series, thus indicating possibility of a feasible approach. Reduction of algae in lagoon effluent is important both from the standpoint of appearance and BOD as well as from the standpoint of interference with effluent disinfection.

The absence of cold weather operating data involving multiple ponds operated in series forced the Board and staff to extrapolate from data showing acceptable final effluent results from 6 to 7 ponds with detentions of around 1 month in warmer climates.

On the basis if information obtained, both from the "experts" and from the literature, and recognizing the need for design standards which would insure reasonable predictability of lagoon effluent quality, design standards were finally developed and adopted which included the following basic requirements:

1. For all except total containment lagoons, there shall be a minimum of 3 cells of approximately equal volume, providing a total theoretical detention time of at least 60 days in summer and 120 days in winter, plus chlorination of the final effluent

   OR

2. Provision of additional cells (at least a total of 6) of approximately equal volume, plus additional detention (approximately 75 to 90 days in summer and 150 to 180 days in winter), in order to eliminate the effluent chlorination requirement.
3. A suggested increase in surface area of the primary pond, where necessary, for control of odors.
4. Limitation of seepage to a maximum of ¼ inch per day.

Utah's lagoon design standards, as developed and as currently constituted, are included as an attachment to this paper.

Because these standards were developed by extrapolation of the very limited information available, Utah always warns those using these standards as a design basis that an actual determination of final effluent quality will determine acceptability of the facilities and that if needed results are not obtained then additional ponds will be required.

Since development and acceptance of Utah's lagoon standards, several lagoon facilities have been constructed, as indicated in Table I but only a limited amount of operational data has been developed to date, since most of the facilities are as yet discharging no effluent. This is in part due to the fact that some have been in operation for a relatively short time and others are obviously plagued with excessive seepage.

A brief summary of limited available performance data on the Logan, Utah lagoon may be in order, however, and is presented in Figure I and Tables II and III.

It is interesting to note that, although BOD_5 and MPN coliforms in the Logan lagoon effluent were well within accepted limites most of the time, there was a radical deterioration, with respect to effluent coliform quality in February and March, 1969. There was also a problem demonstrated in August, 1968 which was not repeated in this time period in 1969. Effluent BODs in excess of 25 mg/l also occurred in some individual samples collected in June and August, 1969.

The use of chlorination for continuous effluent disinfection was anticipated as a need when the Logan lagoon was designed, and may yet be necessary when the ponds become more heavily loaded. In the meantime, effluent chlorination certainly is necessary during periods of high coliform counts, and there may be a need for more frequent sampling to more clearly define these periods.

It is our feeling that application of Utah's lagoon standards to date substantiate the validity of earlier considerations to the effect that parameters of total detention period, multiple series ponds and the possibility of effluent chlorination are necessary to achieve consistent effluent quality in accordance with Utah's standards. It is our further feeling that many avenues of research should be actively pursued, some of which might include the following:

1. To reduce short-circuiting tendencies, ponds downstream from the primary cell might be designed long and narrow. To facilitate such design, inner dikes between ponds might conceivably be constructed more cheaply of light-weight materials since designs can be such that hydraulic pressures are equalized.
2. The effect of long narrow configuration and total number of ponds on final effluent quality should be studied under various weather conditions.
3. The loss of water through seepage and evaporation should be determined with greater precision. It appears that seepage losses may generally be much higher than most experts are willing to acknowledge.
4. Control of algae needs more study—especially the possibility of "starving" algae by use of many ponds. The possibility of killing remaining algae in the final pond might be investigated.

III-82. *Chlorinations*

a. Treatment works effluents shall be chlorinated for reduction of bacteria and viruses as required by final conditions of disposal and as stipulated by the Boards.

b. In general, chlorination will be considered fully effective for the purposes of these regulations when applied to oxidized effluents retaining not more than 25% of the raw wastewater B.O.D. and containing not more than 50 mg/l of B.O.D. and not more than 50 mg/l of suspended solids.

c. The use of equipment designed to feed chlorine gas in solution is recommended.

d. The chlorinator building or room shall have ample forced air ventilation. When necessary, heating facilities shall be installed. If gas chlorination equipment and chlorine containers are to be placed in a building used for other purposes, a tight partition shall separate this room from any other portion of the building. Doors should open only to the outside of the building.

e. Chlorinator capacities required will vary, depending on the use and point of application of the chlorine. For disinfection, the capacity should be adequate to produce a residual of 1.0 mg/l in the final effluent. For normal domestic wastewater effluents, the following dosing capacity will usually be sufficient.

Type of Treatment	*Dosage (Based on Design Average Flow)*
Trickling filters	15 mg/l
Activated sludge	8 mg/l
Sand filters	6 mg/l
Stabilization ponds	20-25 mg/l

TABLE I
LAGOON CONSTRUCTION IN UTAH
TO JUNE, 1970

		DESIGN DATA					Total	
Community	Placed in Operation	Pop. Eq. (BOD$_5$)	Flow MGD	No. Cells in Series	Detention (days)	BOD Load Pri. Cell (#/Acre/day)	Pond Area (Acres)	Remarks
Hurricane	1967	1,100	0.12	3	Total Containment	35	7.5	
Logan	1967	44,700	14.6	5	60(S) 120(W)*	40	465	
Manila	1967	600	0.08	7	180	55	8.6	
Roosevelt	1967	3,600	0.34	6	150	40	29	No effluent discharge - Excessive Seepage
Nephi	1968	3,000	0.3	3	120	709**	20.5	No effluent discharge - Excessive Seepage
Parowan	1968	1,850	0.154	5	Total Containment	40	42	
Duchesne	1969	2,200	0.29	3	120	37	23	
Altamont	Under Const.	160	0.016	3	Total Containment	18	3.0	
Corinne	Under Const.	700	0.070	7	150	40	7.5	
Kamas	Under Const.	1,100	0.11	4	Total Containment	28	14.8	
Morgan	Under Const.	1,650	0.19	7	150	40	19.7	
Plain City	Under Const.	4,200	0.42	3	120	33	40	
Delta	Plans Approved	4,700	0.51	6	150	40	61	
Wendover	Plans Approved	3,000	0.20	6	180	40	33.2	

* S - Summer, W - Winter

** Mechanical Aeration Provided

Cell	Water Surface Area (Acres)	Effective Vol. @ 6' depth
A_1	95.1	24,542,000 ft^3
A_2	95.0	24,532,000 "
B_1	70.9	18,274,000 "
B_2	72.3	18,648,000 "
C	64.4	16,194,000 "
D	39.4	10,104,000 "
E	28.3	7,242,000 "
Total	465.6	119,536,000 ft^3

Flow diagram of
LOGAN, UTAH LAGOON

FIGURE I

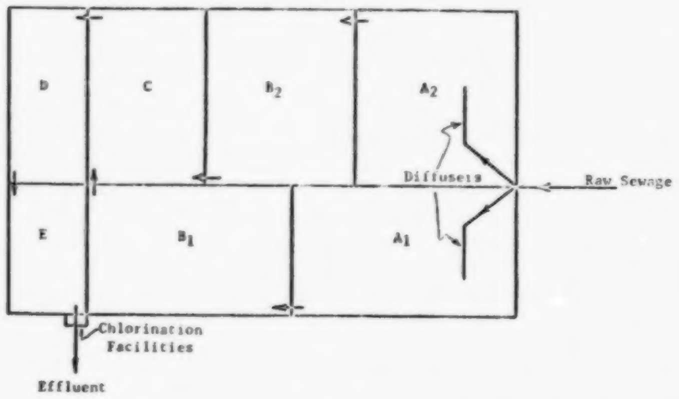


TABLE II
SUMMARY OF LAGOON OPERATING DATA
Logan City, Utah
Aurugt 1968 - October 1969

Based on Weekly Samples & Observation by Utah State University

	No. Samples & Observations	Ave. Flow MGD		MPN Coliforms/100 ml Lagoon Effluent		Biochemical Oxygen Demand (BOD$_5$) - mg/l Raw Sewage		Biochemical Oxygen Demand (BOD$_5$) - mg/l Lagoon Effluent		Ave. Theoretical Detention (days)	Ave. BOD Loading Primary Cells #/Acre/da	Ave. Temp-°C Lagoon Effluent
		Raw	Effl.	Ave	Range	Ave	Range	Ave	Range			
1968												
Aug.	4	13.2	12.5	12,483	230-24,000	26.9	18-39	3.5	0.0-6.6	68	16.7	22.0
Sept.	5	11.0	9.0	98	15-240	83.0	24-158	7.1	0.0-24.0	81	40.0	17.5
Oct.	4	8.3	8.1	31	2-38	73.5	55-109	1.6	0.0-2.9	108	26.8	11.5
Nov.	4	7.2	6.6	4	2-9	109	85-127	2.5	1.0-3.4	124	34.5	5.7
Dec.	5	6.0	5.9	2	< 1-5	156	88-305	4.0	1.4-6.4	149	43.7	3.0
1969												
Jan.	4	7.6	6.8	234	< 1-930	91	67-115	2.4	0.4-3.6	118	30.3	2.1
Feb.	4	7.3	8.8	5,475	900-9,300	89*	79-100	3.9	0.4-6.0	128	27.3	1.6
Mar.	5	5.9	7.8	229,800	43,000-460,000	81	70-94	13.9	6.8-21.6	113	28.1	2.1
Apr.	4	7.1	6.6	60	< 1-230	104	90-115	13.7	6.6-19.2	126	32.5	11.7
May	4	11.5	9.7	3	< 1-5	86	42-106	9.8	5.4-15.0	78	43.6	19.8
June	5	13.3	10.9	13	< 1-38	138	59-218	22.2	11.4-41.9	67	80.6	20.6
July	4	14.5	11.1	120	12-430	88	71-106	12.3	10.8-16.9	62	56.1	25.2
Aug.	4	14.7	14.1	115	38-230	93	20-141	19.0	7.2-37.3	61	59.8	26.0
Sept.	4	12.6	13.4	311	21-750	60	29-124	12.4	4.8-19.3	71	33.3	20.0
Oct.	4	8.7	11.4	15	2-38	69	24-118	5.1	0.0-8.4	103	26.4	9.4

* 2 Samples Only - Deep Snow Prevented Access for Sampling.

Notes:

1. Lagoon Operation Began Nov. 1967 with effluent discharge beginning in March 1968.
2. Chlorination facilities provided but not yet operated.

TABLE III
SUMMARY OF LAGOON OPERATING DATA
Logan City, Utah
September 1968 - September 1969
Based on effluent "grab samples" by Utah State Division of Health

	B.O.D.$_5$	MPN Coliforms/100 ml
September 19, 1968	2.1 mg/l	< 230
October 23, 1968	1.9 mg/l	< 23
December 26, 1968	4.1 mg/l	< 23
March 4, 1969	5.3 mg/l	93,000
April 9, 1969	12.4 mg/l	930
May 21, 1969	3.5 mg/l	430
July 2, 1969	3.7 mg/l	< 230
August 13, 1969	1.5 mg/l	< 230
September 24, 1969	1.7 mg/l	< 230

f. Following thorough mixing, a chlorine contact period of at least 30 minutes, and preferably 60 minutes, shall be provided for disinfection. Contact period shall be calculated at design average flow or maximum rate of pumpage, whichever is greater. The chlorine contact tank shall be designed to reduce short circuiting of flow to a practical minimum. Contact time in an outfall sewer may be honored if desired. Facilities for prevention or easy removal of sludge accumulation should be provided in chlorine detention tanks.

Excerpts from Part III, Utah Code of
Waste Disposal Regulations (1965)

WASTE STABILIZATION PONDS (Lagoons)

III-83. *General*

Waste stabilization ponds may be used for treatment of either raw waste-water or effluent from other treatment processes. Ponds treating domestic wastewater can generally be expected to produce the equivalent of secondary treatment as described in preceding sections when proper attention is given to all design details covered in following paragraphs. However, since pond operation is influenced by somewhat uncontrollable and relatively unpredictable natural phenomena such as temperature, cloud cover, precipitation, soil permeability, etc., adequate factors of safety should be provided in the design. One problem deserving of special attention is the prolific algae production which is necessary in early phases of pond treatment but which may interfere with final effluent quality, especially when chlorination is employed.

The uncertainties involved in prediction of effluent quality may result in a need for construction of additional ponds or other modifications of design after facilities are placed in operation.

III-84. *Isolation*

All ponds receiving domestic wastewater shall be suitably isolated as follows:

a. They shall be located as far as possible from human habitation or from any area which may be built up within a reasonable future period. Such distance of isolation should be not less than 1000 feet.

b. They should be so located that local prevailing winds will blow from the pond area to uninhabited areas. An unobstructed sweep of winds across ponds is desirable.

c. Proximity of ponds to water supplies and other facilities subject to contamination should be critically evaluated to avoid creation of health hazards or other undesirable conditions. The possibility of chemical pollution may merit special consideration. Areas of porous soil and fissured rock formation shall be avoided, unless sealed off to meet requirements of Section III-88.

d. Ponds shall be enclosed within a fence adequate to exclude domestic pets, livestock and people. Access gates with locks shall be provided.

e. Appropriate warning signs should be provided along the fence to designate the nature of the facility and advise against trespassing.

III-85. *Number and Size of Ponds*

Except as indicated elsewhere in this code ponds treating domestic wastewater shall be sized and divided as follows:

a. Total pond capacity based on actual depth not in excess of 6 feet shall provide theoretical wastewater detention of 120 days in winter and 60 days in summer, and such additional detention as found necessary to insure at all times a degree of treatment meeting requirements of this code. (See paragraph "c" below).

b. For the purpose of reducing short circuiting and limiting algae in the final effluent, total pond capacity shall be divided into at least three cells (of approximately equal capacity) designed for series operation. Division of total capacity into more than three cells is recommended in any case and is required under the conditions of paragraph "e" below. Any such cell may be subdivided into sub-cells designed for parallel operation to further reduce short circuiting.

c. Capacity of primary cells determined by the method of paragraph "b", above, should be increased when necessary to assure absence of odor nuisances. Loadings of less than 40 pounds of B.O.D. per acre per day are considered necessary for this purpose.

d. Where chlorination is included as part of the overall treatment process it shall be accomplished by facilities conforming to the requirements of Section III-82.

e. If chlorination is not proposed, adequate reduction of infectious agents and other contaminants shall be assured through increased total capacity (detention time) and through division of such increased total capacity into more than three cells by the method of paragraph "b", above.

f. Storage throughout the winter season may be used in lieu of increased detention provided pond overflow device is so designed as to produce a discharge rate approximately proportional to pond depth, not exceeding Q at minimum pond depth or 2 Q at twice minimum depth or more, where Q is design average flow. Due consideration shall be given to the ability of receiving streams to accept the increased discharge rate.

g. Due consideration should be given to future population growth and other factors which might influence quality, quantity and strength of wastewater: Suitable land should be available at the site for increasing the size of the original construction.

h. Quantity and type of all industrial wastes present in the wastewater must be identified and given due consideration.

i. All cells shall be designed for minimum liquid depth of three feet to discourage development of emergent vegetation and to aid thereby in mosquito control.

j. If ponds are to receive partially treated effluents, specific design requirements will be established performance of ponds under these circumstances.

III-86. *Non-overflow Ponds*

Ponds designed for retention of contents without surface overflow shall have sufficient size to assure this condition of the basis of expected evaporation loss with full consideration of storage needs during critical winter months. They shall be provided with emergency spillways for dike protection. Activation of such spillways will constitute evidence of need for additional ponds.

III-87. *Embankments and Dikes*

a. Embankments and dikes shall be constructed of relatively impervious materials. Compaction and other features of construction and design shall insure a stable structure. Vegetation should be removed from the area upon which the embankment is to be placed.

b. Embankments and dikes shall be so placed as to avoid narrow or elongated sections in ponds and also to limit short-circuiting. No islands, peninsulas or coves should be permitted. Dikes should be rounded at corners to minimize accumulations of floating materials.

c. Top width of the embankment shall be at least eight feet to permit access of maintenance vehicles.

d. Embankment slopes should have a ratio of horizontal to vertical distance of approximately three. Riprapping may sometimes be necessary for inner slopes to insure stability, especially for large ponds. Flat slopes have the disadvantage of encouraging emergent vegetation, which aggravates mosquito control problems.

e. Minimum freeboard between maximum water level and embankment top shall be three feet except for very small installations.

f. In general, seeding of embankments is not practical in Utah due to low rainfall, and therefore other erosion control measures may be necessary.

III-88. *Pond Bottom*

a. The pond bottom should be as level as possible at all points but may have a slight uniform slope where justification can be shown.

b. The bottom should be cleared of vegetation and debris. Organic material thus removed should not be used in the dike core construction. However, suitable topsoil relatively free of debris may be used as cover material on the outer slopes of the embankment.

c. Because maintenance of a satisfactory water level in ponds is an important aspect of design and successful operation, the soil formation or structure of the bottom should be relatively tight to avoid excessive liquid loss due to seepage. Soil borings and tests to determine the characteristics of surface soil and subsoil shall be made a part of preliminary surveys to select pond sites. Gravel and limestone areas shall be avoided unless effective and permanent sealing is accomplished. Removal of porous topsoil and proper compaction of subsoil improve the water-holding characteristics of the bottom. Removal of porous areas, as gravel or sandy pockets, and replacement with well-compacted clay or other suitable material may be indicated.

d. Where seepage is determined to be in excess of ¼" per day, sealing of the bottom with clay, bentonite, asphaltic coating, or other sealing material should be given consideration. In areas where ground water is a source of water supply sealing of ponds will be required.

III-89. *Inlet and Outlet Structures*

a. All influent and effluent structures should be located to minimize short-circuiting within ponds, and to avoid blocking of pond circulation. Such structures must have protection against freezing or ice damage under winter conditions.

b. Primary cell inlets shall meet the following design criteria.

(1) Surcharging of the sewer upstream from the inlet manhole is not permitted.

(2) The influent line shall be located along the bottom of the pond with the top of the pipe just below average elevation of the pond bottom. This line can be placed at zero slope.

(3) Discharge shall be essentially in the center of a round or square cell, and approximately at the third point farthest from the outlet structure in a rectangular cell.

(4) Gravity flow inlets should discharge horizontally into a shallow, saucer-shaped depression extending below the pond bottom not more than the diameter of the influent pipe plus one foot.

(5) Force mains shall be valved at the pond, and may terminate in either vertically upward or horizontally discharging sections, but the discharge end of vertical pipes should never be located more than one foot above the pond bottom, because of ice damage possibilities. Flow velocities should be in excess of two feet per second.

c. The discharge end of pipes should rest on a suitable concrete apron with a minimum size of two feet square.

d. At points where pipes pass through embankments, manholes or cleanouts should be installed to facilitate sampling, inspection, and maintenance.

e. Outlet structures for all cells shall consist of a manhole, box or other suitable device, which may be adjustable in the case of ponds designed under requirements of Section III-85-f. Minimum outlet level shall be at least three feet above pond bottom in any case. When possible, the outlet structure should be located on the windward side to prevent short-circuiting.

f. Piping should be designed for complete regulation of flow between cells, and in such manner as to minimize need for erosion control measures.

III-90. *Flow Measurement*

Adequate flow measurement devices shall be provided for all installations.

QUALITY PERFORMANCE OF WASTE STABILIZATION LAGOONS IN MICHIGAN

MAURICE S. RICHMOND
Sanitary Engineer
Michigan State Department of Health

It is considered that a presentation of the Quality of Performance of Lagoons in Michigan requires a short synopsis of the historical development of the use of lagoons, the manner in which they are designed, constructed and operated, and requirements of the receiving waters. It is hoped that out of this framework, the data and its analysis will take on more specific meaning.

Historical

The evolution of waste stabilization lagoons as a recognized method of treating raw wastewaters in the midwest is outlined in some detail in the proceedings of the Kansas City Symposium held in 1960. Michigan's first municipal lagoons were constructed the following year (in 1961) at Lawton, in the southwestern corner of the state. Today, 62 Michigan communities operate waste stabilization lagoons, 16 more are under construction and 115 are being planned for construction within the next two years. These are quite well distributed throughout the state as shown in Figure 1. Operated with a minimum of nuisance and providing effective pollution control, these have become increasingly popular in Michigan. The rather general, and sometimes bitter, opposition to lagoons experienced in the early days has turned to ardent support. In the beginning, advocates were few and critics many. Today we would be hard pressed to find anyone, be it a consulting engineer or a public official, denying that these lagoons have virtue.

Uses of Waters Requiring Protection

In Michigan, pollution control facilities, in addition to protecting a multitude of domestic and industrial water supplies, are essential to the state's vast outdoor recreation program. Some indicators of the magnitude of outdoor recreation in Michigan are the state's 36,000 miles of rivers and streams, nearly one-half million registered motor boats and a tourist business volume, which last year exceeded 1-1/3 billion dollars. The predominating situations in which lagoons have been installed or are planned in Michigan, are those requiring a high degree of treatment because of limited stream capacity and high water quality standards.

Policy Regarding Phosphorus Removal

As reported by the author at the 1966 annual meeting of the Water Pollution Control Federation, an analysis of Michigan lagoon data had indicated that under existing conditions of design and management, lagoons did not dependably remove substantial amounts of soluble ortho-phosphate. Data developed since 1966 has indicated in an equally conclusive manner that these systems cannot be depended upon for a uniformly substantial removal of total phosphorus. With these facts in mind, the Michigan Water Resources Commission has adopted a policy regarding the use of lagoons for wastewater treatment consistent with the water quality standards which, in general, require the removal of at least 80% of total phosphorus from wastewater. This policy recognizes that the total accumulated loading on all present and proposed lagoons in Michigan represents only approximately 2% of the domestic wastewater to be treated and disposed of in the state. On this basis, the use of lagoons is approved, except where the discharge is to be into or just upstream from a lake or impoundment. This policy recognizes the overriding advantages of complete retention of wastewater during the recreation season as compared to the minimal amount of phosphorus discharged by these systems.

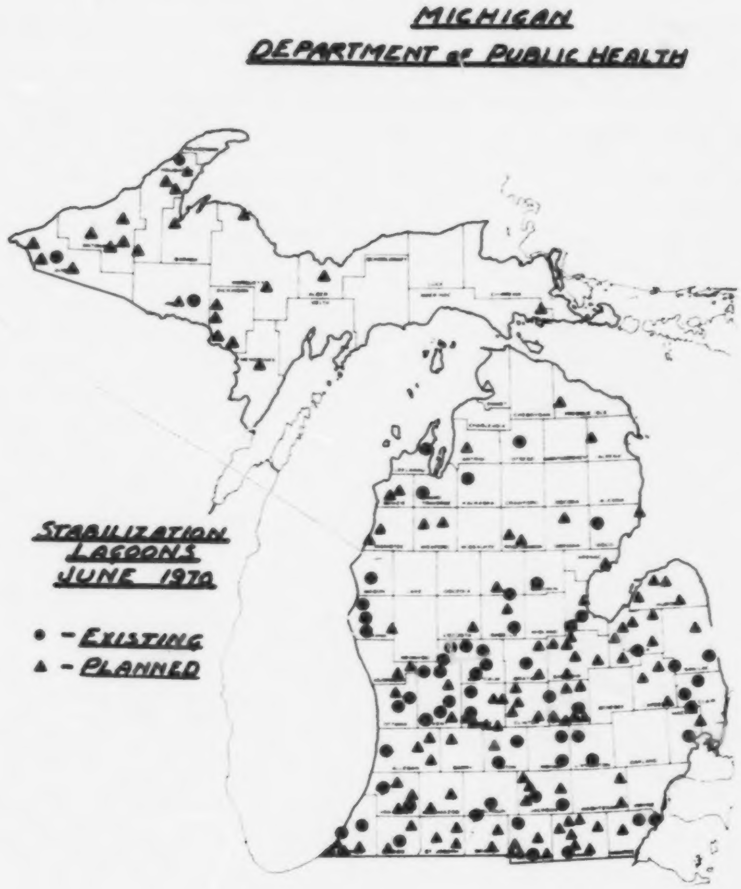
Lagoons are especially adapted to these circumstances by reason of:

1. Their lower initial capital cost;
2. Their lower operational cost;
3. They can be operated by the usual public works staff with less technical competence than that required for more sophisticated treatment facilities; and
4. The maximum possible degree of treatment can be provided by complete retention during critical seasons of the year, with no pollutional effects whatsoever during recreational-high stream use periods.

In this framework, we have adapted the Design Standards recommended by the Great Lakes—Upper Mississippi River Board of State Sanitary Engineers and the Missouri River Basin State Sanitary Engineers, to the Michigan situation.

Design and Construction

A major consideration in the design of any wastewater treatment facility is *loading*. By adopting the rather



conservative northern latitude loading limits so well publicized, essentially 1 acre of lagoon surface per 100 people served and 20 lb. of BOD$_5$/acre/day, it is possible to provide for complete retention of wastewater during all but about two weeks in the fall and two weeks in the spring. Thus, we meet the highly desirable objective of having no discharge at all during the season of the year when maximum protection to surface waters is so badly needed but seldom obtained.

A lagoon system, capable of storage through the long spring-summer-fall period, has plenty of capacity through the winter period when the lagoon process is least effective by reason of ice and snow cover, drastically lower water temperatures and reduced effects of sunlight. In fact, sufficient storage is available to extend the period of retention long enough beyond the time when the ice goes out to permit nature to establish a water quality in the lagoons quite comparaboe to that discharged in the fall.

To accomplish all this, the lagoon facility must have:

1. A capability of controlling water levels through a somewhat wider than usual range;
2. Controlled drawoff capability;
3. Flexibility permitting feeding, or withdrawal from, any single cell.

Generally speaking, all of the recommended features of design in the Great Lakes—Upper Mississippi River Board of State Sanitary engineers (the so—called 10 States) standards, are adhered to quite rigidly. Particular attention has been given to grading and compaction of soils on both bottoms and side slopes. A serviceable roadway on the dykes capable of accommodating trucks and other maintenacne equipment is considered essential. We have a wide variety of soils in Michigan, ranging from very sandy, usually in the recreational areas of the state, to a variety of loams with clay admixtures in the agricultural areas. Except for two rather unsatisfactory experiences with bituminous coatings, the sealing of under water surfaces in Michigan lagoons has been accomplished with the use of clay admixtures. Although both of the two installations where bituminous coatings were used were unsatisfactory, the Asphalt Institute has expressed great interest in developing both specific products and methods of application to provide an enduring, effective membrane for sealing underwater lagoon surfaces. Manufacturers of other membrane materials have likewise expressed interest in furnishing a sealing material for lagoons, but as yet none have demonstrated in practice in Michigan municipal systems. Multiple lagoon cells are required uniformly. We have learned from experience that a good deal of flexibility is required for effective performance. In our opinion, such flexibility must permit both parallel and series operation, isolation of any one cell and drawoff from any selected level. We are now requiring maximum flexibility of this nature in all new lagoon systems, and where not originally provided, it has been installed in several older systems.

We believe that flow measurement is essential in all installations. We need it to determine the rate and magnitude of the load. It is especially useful in determining the quantity of ground water and surface runoff admitted to the sewer system and has other obvious values such as identification of, or determination of, the amount of organic loadings.

Management–Routine Operation and Maintenance

On each week day, a regular member of the municipal public works staff drives to the lagoon site, usually in a small truck. He unlocks the gate (all lagoon systems in Michigan are required to be well fenced and equipped with locked gates) and then drives along the dykes, with windows open, traveling slowly all the way around each lagoon cell. During this drive, he inspects the dykes for erosion, damage by burrowing animals, need for mowing grass and any other maintenance required. He also is checking for scum accumulations, and odors.

After this tour of inspection, he notes and records the water surface elevation (ice thickness in winter periods), records the flow meter data and pertinent climatic information such as wind direction, whether cloudy or sunny and air temperature. These records are entered on forms furnished by the Department of Public Health, as shown in Figure 9 (a).

Any maintenance work required is scheduled and performed as soon as the urgency of the situation demands. Quite often, grass mowing is performed by persons employed specifically for this type of work in other areas of the community, such as the parks and cemeteries. Mowing of grass on the side slopes of the dykes presented the most troublesome aspect of lagoon maintenance in the early days of Michigan lagoon experience. Special equipment with a low center of gravity has been adapted for most installations which has provided a reasonably successful solution to this problem.

Management–During Discharge

The Engineering Staff of the Michigan Department of Public Health has assumed responsibility for making the overall decision as to when, at what rate, and how much to discharge from all municipal lagoons and for the establishment of the detailed discharge control program. Each fall in October, and again in the spring before the ice break-up, a Health Department basin engineer visits each lagoon system, inspecting the lagoon, the receiving stream and the routine records mentioned earlier. Where the conditions and records indicate that capacity reserves are insufficient to provide retention until the next possible discharge period, it is determined that a discharge will be needed. He goes over once again the detailed instructions which the operator has received each season, checks for an adequate supply of sample containers, and in general makes sure everything is in readiness for the forthcoming discharge. A subsequent inspection is made at the time a discharge should be made, in late October for the fall discharge, and again after the ice break-up in the spring for the spring discharge. At this time, dissolved oxygen is determined, both in the lagoons and in the receiving stream. The flow and temperature in the stream are also determined. With this information in hand, the basin engineer then lays out a detailed discharge program for the local person in responsible charge.

This program requires control of rate of discharge and an intensive sampling routine. Samples of the lagoon discharge are composited by the operator and delivered by pre-arrangement to a nearby wastewater treatment plant laboratory for analysis under a contractural arrangement. Currently these composites are analyzed for pH, BOD_5, suspended solids and volatile suspended solids. Grab samples are collected each day during the discharge period for both total and fecal coliform analysis. A tabulation indicating the magnitude of this sampling and testing program is shown in Figure 2. Records of these analyses are filed after each discharge on forms furnished by the Michigan Department of Public Health. (See Figure 9 (c).

While the first lagoon in Michigan was built at Lawton in 1961, that system has not yet been sufficiently loaded to require a discharge. Other systems did require a very limited discharge by the fall of 1964 and the first full fledged discharges occurred in the spring of 1965. An increasing number of lagoon systems have been discharged each spring and fall since then. For comparative purposes, records of lagoon systems which have been discharged each season beginning with 1965 are included in the data presented later.

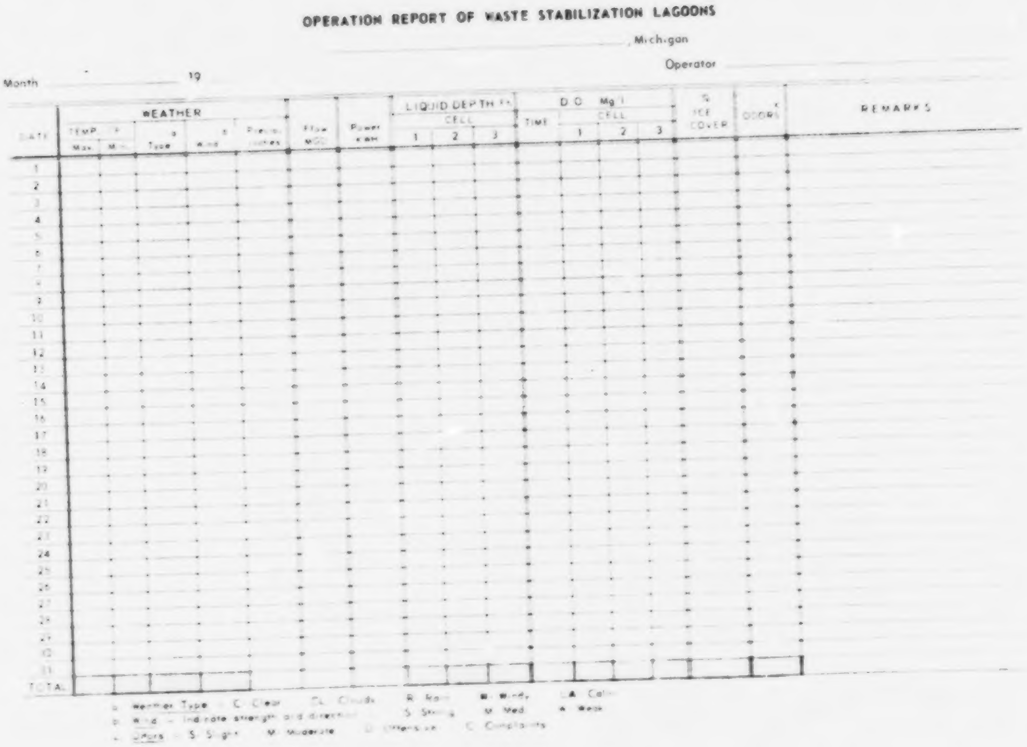
Although only the performance data from those lagoons where a sufficient number of samples were collected to permit statistical analysis are reported here, similar data from all of the others has been thoroughly studied and have been found to conform in all essential respects to the data from these systems.

Performance–Physical Conditions

While some temporary operational problems, resulting in localized odors have occurred, no serious odor nuisance conditions have been observed or reported from Michigan municipal lagoons thus far, except following the spring ice break-up. During the spring break-up periods, this spring, for the first time we observed odors in the immediate vicinity of some of the lagoons which persisted in 2 or 3 cases for as long as 21 days.

In two lagoon systems, at Fowlerville and Bangor, profuse growths of filamentous algae have developed at times forming what appeared to be solid banks some three feet or so in depth over about three fourths of the lagoon area, having the appearance of green icebergs suspended in the water. These eventually began to decompose, with extensive floating masses appearing on the lagoon surface. These decaying masses of algae have, of course, given off odors, although these were confined to the immediate vicinity of the lagoons. In both of these cases, the loadings were very low and at Fowlerville, the depth was only about 18 inches just preceding the heavy filamentous algae blooms thus permitting greater than normal sunlight penetration into the liquid. Normal conditions were restored when the decaying material either settled to the bottom or was removed physically, and this is not a simple operation.

Generally speaking, with the short period exceptions mentioned, the municipal waste stabilization lagoons in Michigan present an attractive appearance, looking to the average person like a series of artistically arranged ponds in a natural park setting. Many of these have sizable flocks of ducks–both nesting and migratory. In and around communities where lagoons have been built, there has been



OPERATION REPORT OF WASTE STABILIZATION LAGOONS

__________________, Michigan

Operator __________________

Month __________________ 19

DATE	WEATHER					Flow MGD	Power KWH	LIQUID DEPTH Ft.			D.O. Mg/l				% ICE COVER	ODORS c	REMARKS
	TEMP. °F		a	b	Precip. inches			CELL			TIME	CELL					
	Max.	Min.	Type	Wind				1	2	3		1	2	3			
1																	
2																	
3																	
4																	
5																	
6																	
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25																	
26																	
27																	
28																	
29																	
30																	
31																	
TOTAL																	

a. Weather Type — C Clear CL Cloudy R Rain W Windy CA Calm

b. Wind — Indicate strength and direction S Strong M Med W Weak

c. Odors — S Slight M Moderate O Offensive C Complaints

FIG. 9(a)

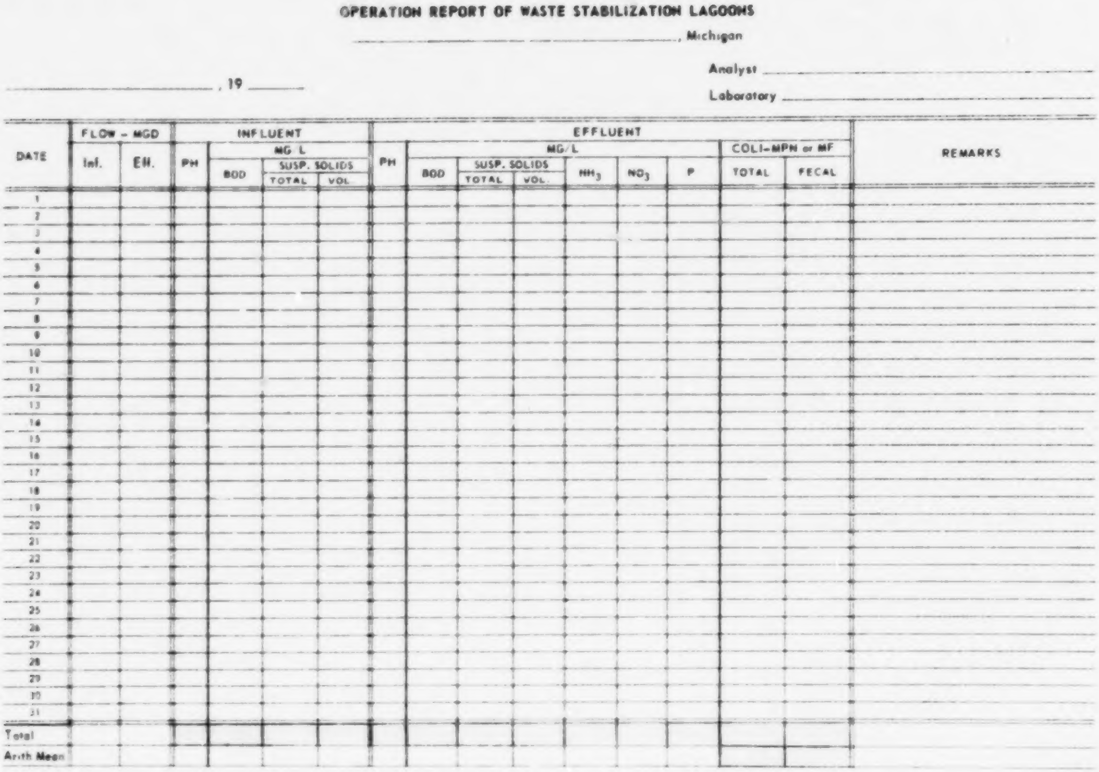
LAGOON EFFLUENT- SAMPLING & TESTING
MICHIGAN DEPARTMENT OF PUBLIC HEALTH

	SPRING 1965	FALL 1965	SPRING 1969	FALL 1969
NUMBER OF LAGOONS SAMPLED	9	8	18 A	18 B
NUMBER OF DAYS SAMPLED (AVE.)	14	14	13	10
NUMBER OF SAMPLES ANALYZED				
CHEMICAL	122	134	230	184
BACTERIOLOGICAL	145	105	260	187
AREA OF LAGOONS IN ACRES (AVE.)	14	14	16	16
" " " " " (RANGE)	6-36	6-36	4-36	4-36

A-4 OF 18 LAGOONS DID NOT DISCHARGE - FINAL CELL SAMPLED

B-2 " " " " " - " " "

FIG. 2



OPERATION REPORT OF WASTE STABILIZATION LAGOONS

______________________, Michigan

______________________, 19 ______

Analyst ______________________

Laboratory ______________________

DATE	FLOW - MGD		INFLUENT				EFFLUENT									REMARKS
	Inf.	Eff.	PH	MG/L			PH	MG/L						COLI-MPN or MF		
				BOD	SUSP. SOLIDS			BOD	SUSP. SOLIDS		NH_3	NO_3	P	TOTAL	FECAL	
					TOTAL	VOL.			TOTAL	VOL.						
1																
2																
3																
4																
5																
6																
7																
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22																
23																
24																
25																
26																
27																
28																
29																
30																
31																
Total																
Arith Mean																

FIG. 9(c)

good acceptance by the public. In some instances strong opposition expressed by residents and property owners prior to construction of the lagoons, has turned to acceptance and admiration since the systems have been in operation.

Effluent Quality

Individual data for each seasonal discharge have been plotted for each of the lagoon systems mentioned above, (where the discharge extended over a period of days long enough to permit collection of enough samples to permit graphical statistical analysis according to the Velz graphical probability method.) From these curves, the statistical mean values for each constituent analyzed were determined for each discharge period at each installation. The curves in Figures 5 and 6 are typical of those developed by plotting these mean values. These curves indicate the range of probably mean values for these constituents which may be expected from similar systems.

Figure 8 (total coliform data) represents direct probability plotting of individual data from all lagoon systems. No significant difference in effluent quality has been noted when comparing the records of those systems operated in parallel with those operated in series.

During recent months all samples analyzed for total coliform bacteria were also analyzed for fecal coliforms.

BOD_5

Tests for BOD_5 were made on unfiltered effluent samples, incubated in the absence of light in accordance with Standard Methods. On Figure 6 we have plotted the statistical mean BOD_5 values of those lagoon systems having a sufficient number of samples to permit such plotting for the two 1965 discharge periods and for the two 1969 discharges. These were derived from the statistical plots of all such data for each lagoon each of these seasons. Here we see that the most probably BOD_5 concentration for these and similar lagoons is somewhere between 9 and 19 mg/l depending on the time of the year. Examination of the individual data for these lagoons reveals that the BOD_5 concentration is less than 40 mg/l ninety percent of the time. It is interesting to note that BOD_5 values were somewhat lower in the fall discharge seasons than in the spring seasons. Data for the intervening seasons between 1965 and 1969 is quite similar.

An analysis was made of the degree of BOD_5 reduction in two typical systems, one serving a community of 2000 people, the other 1200 people. Effluent BOD_5 in pounds was determined from measured flow and BOD_5 concentration. Influent BOD_5 in pounds was calculated from measured input flow and a per capita BOD_5 contribution to the system estimated at 0.167 lb/capita.

Following is a tabulation of the results of these determinations:

	BOD_5 Reduction	
	Spring-Summer-Fall Period	Fall-Winter-Spring Period
System "A" – (2000 population)	93%	91%
System "B" – (1200 population)	98%	90%

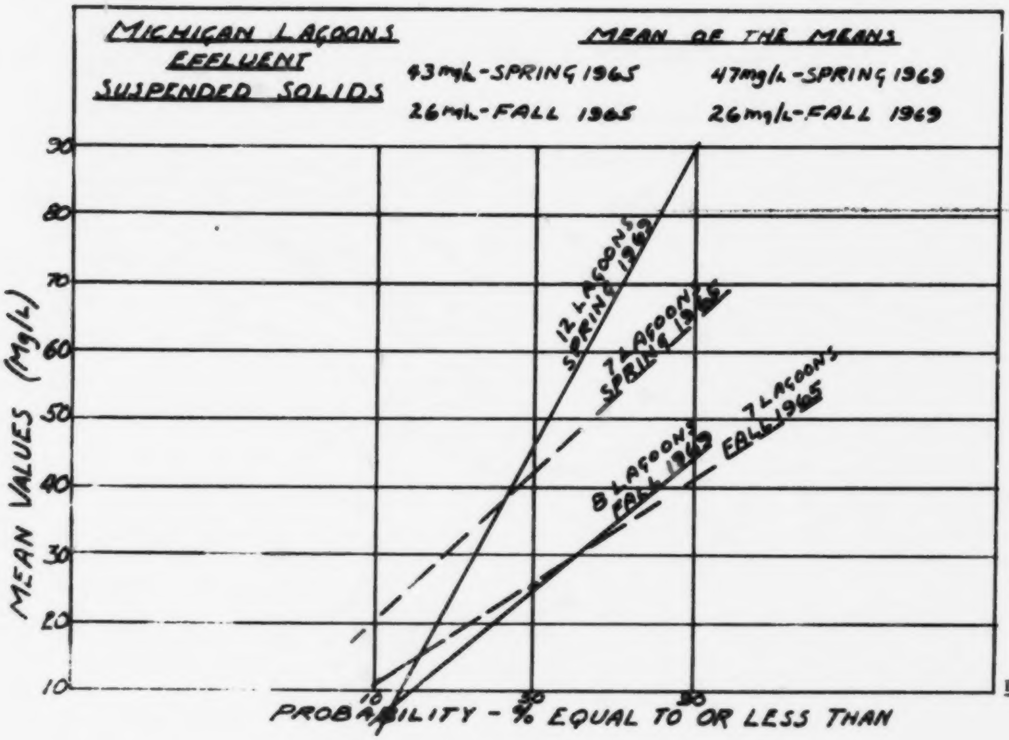

MICHIGAN LAGOONS
EFFLUENT
SUSPENDED SOLIDS
MEAN OF THE MEANS
43mg/L-SPRING 1965
47mg/L-SPRING 1969
26mg/L-FALL 1965
26mg/L-FALL 1969
90
80
70
60
50
40
30
20
10
MEAN VALUES (Mg/L)
12 LAGOONS SPRING 1969
7 LAGOONS SPRING 1965
8 LAGOONS FALL 1969
7 LAGOONS FALL 1965
10
50
90
PROBABILITY - % EQUAL TO OR LESS THAN

FIG. 5

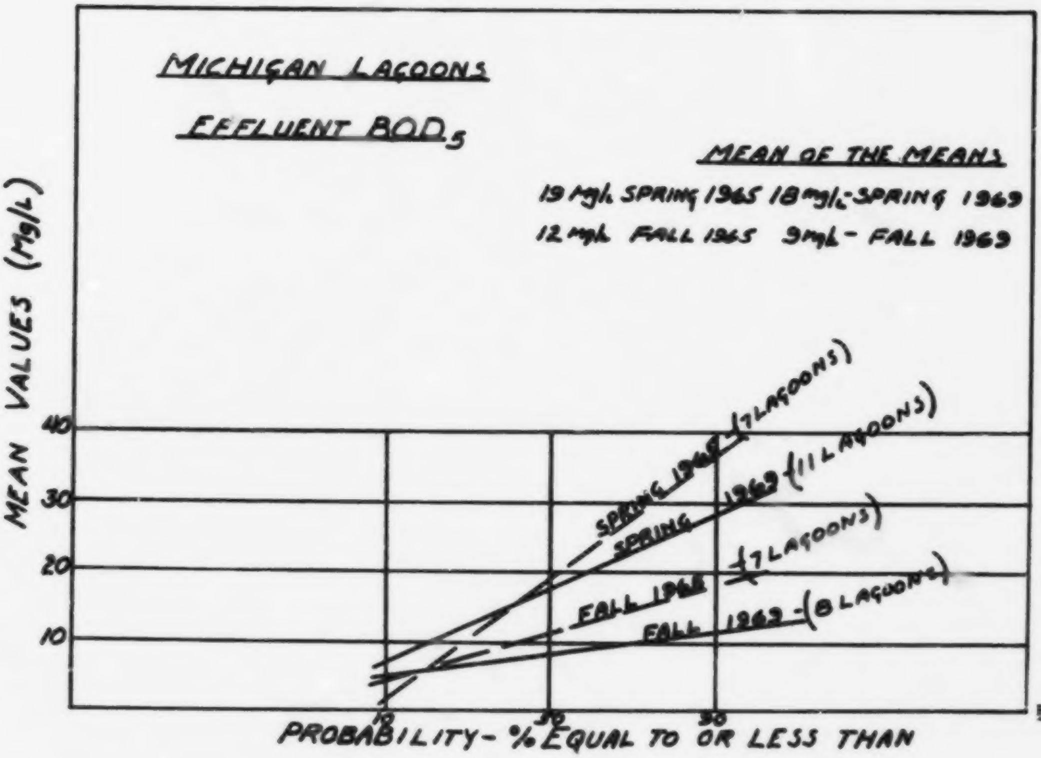

MICHIGAN LAGOONS
EFFLUENT BOD5
MEAN OF THE MEANS
19 mg/L SPRING 1965 18mg/L-SPRING 1969
12 mg/L FALL 1965 9mg/L - FALL 1969
MEAN VALUES (Mg/L)
40
30
20
10
SPRING 1965 (7 LAGOONS)
SPRING 1969 (11 LAGOONS)
FALL 1965 (7 LAGOONS)
FALL 1969 - (8 LAGOONS)
10
50
90
PROBABILITY - % EQUAL TO OR LESS THAN

FIG. 6

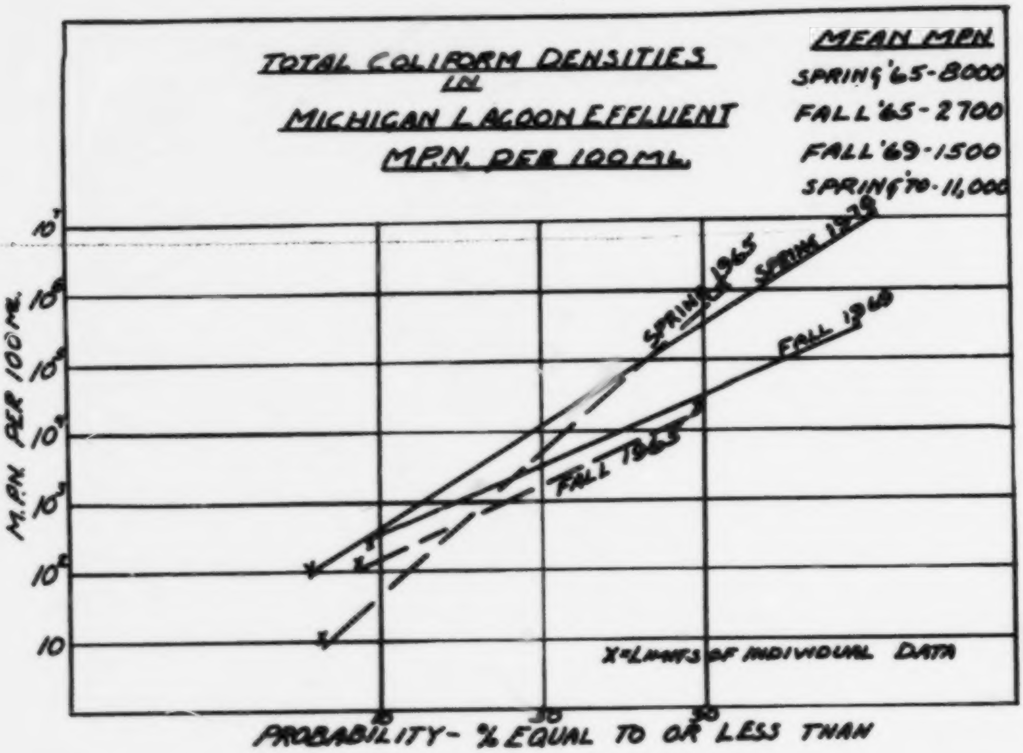


FIG. 8

The effluent BOD5 data from all those systems operated in parallel has been compared with the data from those operated in series. While there are some differences, these were not shown to be significant when applying the overlapping test with as little as two standard errors.

The BOD5 data from two physically quite similar systems, one operated in series, the other in parallel, is shown in Figure 4. It is observed that the mean BOD5 for each system during both a fall and a spring discharge period are 19 mg/l or less.

Suspended Solids

In Figure 5 we have plotted suspended solids data in a manner similar to that used in plotting the BOD 5 data in Figure 6. The most probable suspended solids values for these and similar installations is somewhere between 26 and 47 mg/l depending on the time of the year. Examination of the individual data for these lagoons reveals that the suspended solids concentrations is less than 91 mg/l ninety percent of the time. Again we note that the values were somewhat lower in the fall discharges than in the spring seasons.

A typical relationship between effluent volatile solids concentrations and BOD5 is shown in Figure 7. The records from all lagoon systems studied show similar volatile suspended solids-BOD5 relationships.

Coliform Concentrations

Total Coliform

Figure 8 shows the probability distribution by percentage of time of coliform concentrations in all Michigan lagoon systems. The mean number of coliforms per 100 ml of all samples analyzed for Spring 1965 was 8,000 and for Fall 1965 was 2,700. Likewise, the mean for Fall 1969 was 1500 and for Spring 1970 was 11,000. Individual values range between 10 and 10^7/100 ml.

Fecal Coliform

Figure 3 shows the number of samples analyzed for fecal coliforms in 1969 and 1970 and the percentage of those samples not exceeding certain prescribed levels. It will be noted that the data collected during the 1970 spring discharge season has been grouped in two manners. First, all data collected is set forth. This included some 49 samples collected in the early part of the season while several lagoons were not yet quite severely anaerobic.

These 49 samples were then removed from the total data collection and the remainder, representing those collected from aerobic lagoons, is set forth as noted in figure 3.

It is interesting that this year, with some increase in emphasis on discharge management, the fecal coliforms per 100 ml did not exceed 100/100 ml in 2/3 of the total samples and in ¾ of those collected under aerobic conditions. Also from 80 to 90 percent of the time the samples did not exceed 1000/100 ml during the past two years since testing for fecal coliform bacteria was begun. The percentage of samples not exceeding 5000 fecal coliforms per 100 ml is not substantially higher than the percentage not exceeding 1000/100 ml.

Comparison of fecal coliform data from those lagoons operated in parallel with data from those operated in series fails to show any real differences.

Evaporation—Percolation

Reference has been made earlier in this paper to the analysis of BOD5 reduction in two typical lagoon systems involving measured inflow and outflow. A further analysis reveals a substantial differential between the liquid volumes pumped into the lagoons and those withdrawn in the discharges. This differential is attributed to the combined result of the effects of rainfall upon the lagoon surface,

FECAL COLIFORM DENSITIES
IN MICHIGAN LAGOON EFFLUENTS
COLONIES PER 100 ML.

YEAR	SEASON	NO. LAG. DISCH.	TOTAL NO. SAMPLES	% OF SAMPLES NOT EXCEEDING EFFLUENT DENSITIES SPECIFIED				
				10	100	200	500	1000
1969	SPRING	14	200	0.5	74	77	86	90
1969	FALL	16	162	14	52	60	71	80
1970	SPRING	21	324	47	66	72	77	82
1970	SPRING*	19	275	55	78	84	90	94

FIG. 3

* SAMPLES COLLECTED ONLY WHEN LAGOON BEING DISCHARGED WAS AEROBIC.

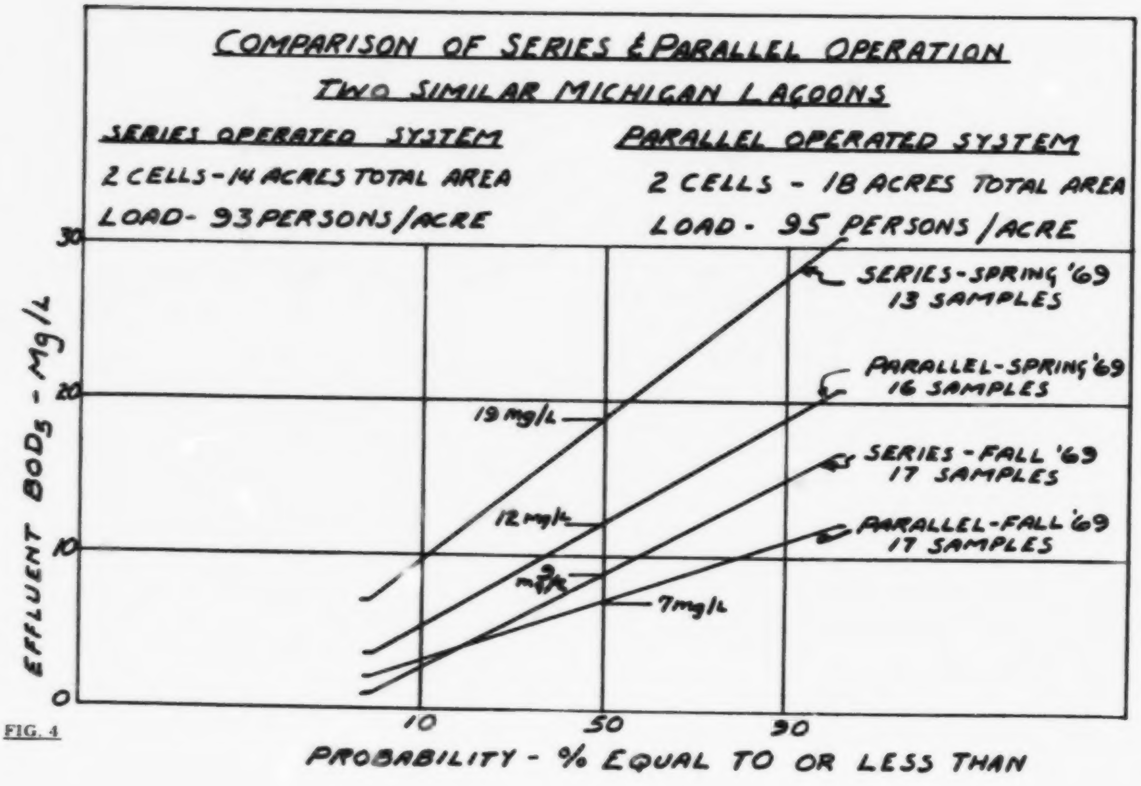


FIG. 4

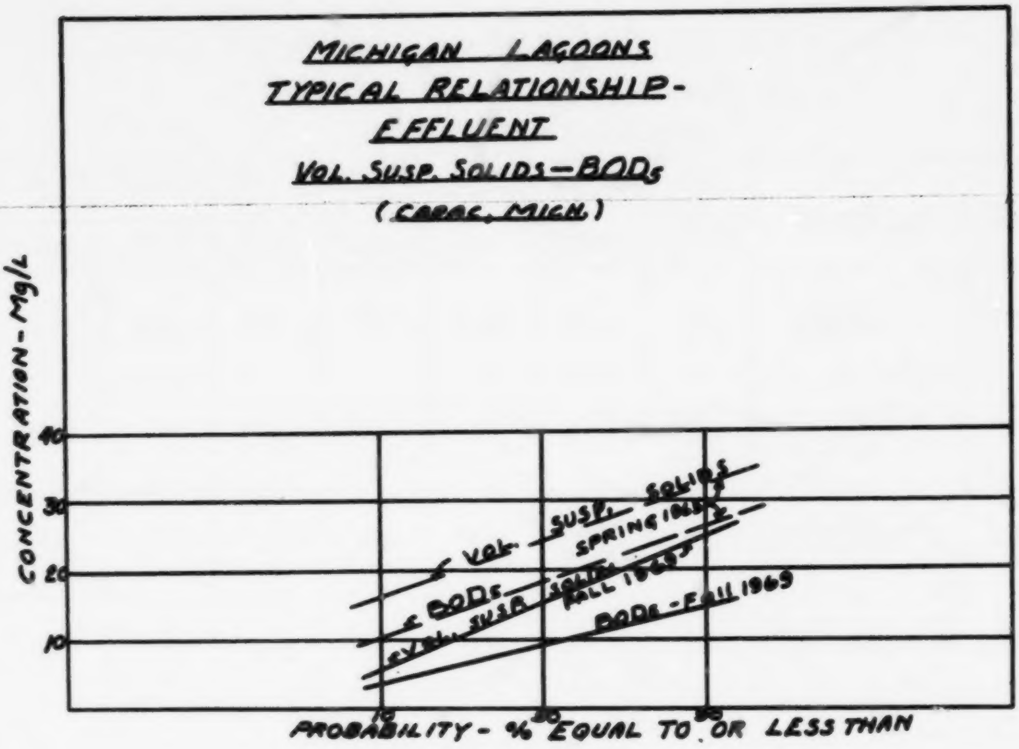
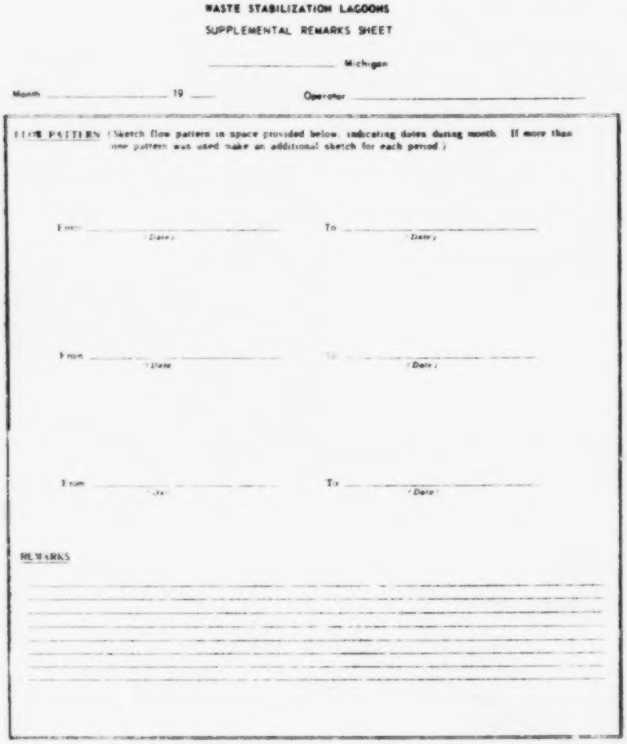


FIG. 7


WASTE STABILIZATION LAGOONS

SUPPLEMENTAL REMARKS SHEET

______________ Michigan

Month ______________ 19 ___ Operator ______________

FLOW PATTERN (Sketch flow pattern in space provided below, indicating dates during month. If more than one pattern was used make an additional sketch for each period.)

From ______________ (Date) To ______________ (Date)

From ______________ (Date) To ______________ (Date)

From ______________ (Date) To ______________ (Date)

REMARKS

FIG. 9(b)

evaporation and percolation into the ground. In the lagoon system serving a community of 2000 in the southeastern part of the state, this differential, or net loss in liquid, averaged approximately ½ inch per day throughout the year. In the second system, serving a community of 1200 in the eastern, or "thumb" area of Michigan, the net liquid loss during the fall–winter–spring period averaged ½ inch per day, while during the spring–summer–fall period, it averaged 0.2 inch per day.

In view of the meager net annual evaporation normally experienced in the Great Lakes Region, a substantial portion of these losses must be attributed to percolation into the soil. This confirms the need for rather exhaustive studies of ground water movement in the vicinity of proposed lagoon sites prior to approval which we have consistently required as protection for domestic ground water supplies.

Summary and Conclusions

In summary, a review of the performance of lagoons in Michigan during the past nine years has been made to determine lagoon effectiveness in general, and more specifically, optimum conditions with respect to design and operational control under Michigan climatic conditions.

All Michigan lagoons are designed for total retention of flow for about six months, with controlled discharge twice a year, once in the late fall and again in the early spring. A concentrated program of sampling and analysis is carried out at each lagoon system during each discharge period. Parameters used in determining effluent quality include DO, BOD, TSS, VSS, pH, total coliform, and fecal coliform.

In general this study reveals that wastewater treatment by the stabilization lagoon method is at least equivalent to that provided by conventional secondary treatment, including removal or reduction of BOD, TSS and coliforms. Effluents discharged during the fall appear to be of somewhat higher quality than those discharged in the spring. The variations in degree of treatment within certain of the systems studied confirm the soundness of the program of discharging to the receiving waters only during the late fall and early spring when the capability of the receiving waters to assimilate the treated wastes is at an optimum level. In addition, the complete elimination of pollution during the extensive period of total retention provides a maximum of protection to the public health and other water use interests.

EVALUATION OF LAGOON PERFORMANCE IN LIGHT OF 1965 WATER QUALITY ACT

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I INTRODUCTION

In 1950 the population of the United States was 150 million, today the population is 190 million, and by 1980 the population has been projected to reach 250 million (1). Because of increased urbanization and industrial expansion, water pollution is a problem that is increasing at an even greater rate than the human population (2). The per-capita water use and wastewater discharge has increased fourfold since 1900 (1).

Since 1900, industrial water use and wastewater discharge in the United States has increased elevenfold to 160 billion gallons per day. In addition, agricultural water use in the United States has increased sevenfold to 145 billion gallons per day (2). Continued pollution, regardless of the source, means less and less water of suitable quality for necessary as well as desirable purposes. The only practical, long-range solution to the problem is wastewater treatment and re-use (2).

Most of the United States population today utilizes raw water supplies that contain water which has been previously used for industrial or municipal purposes or both (2). For cities in the upper 10-percent re-use bracket, their supplies have been previoulsy used at least three times (2). This re-use occurs from community to community as water flows downstream. The quality of this used and re-used water is dependent, in part, on the wastewater treatment processes employed.

The treatment of wastewater in oxidation ponds has gained wide acceptance during the past three decades (3,4). Concomitant with the accelerated increase in the number of oxidation ponds, are reports of a variety of problems (5,6). These periodic problems associated with oxidation ponds have been reported to be:

1. Obnoxious hydrogen sulfide odors.
3. Highly-colored, algae-laden effluents.
4. Malodorous algae blooms and "pig-pen" odors.
5. Excessive amounts of organic matter in lagoon effluents.
6. Noxious vegetative growth.
7. Mosquito and insect breeding.
8. Septic sewage odors.

These problesm often lead to effluent and aesthetic quality failures.

Typical aesthetic failures are often attributed to excessive amounts of undersirable algae (5,6). A typical case of algae accumulation on the shore of a eutrophic lake may be seen in *Figure 1*. This Figure, titled "Showdown," was obtained from the Federal Water Pollution Administration bulletin CWA-11 (7). The caption reads, "A green swath of algae, the cancer of water pollution, rims the rocky beach of a dying lake." It is evident that the obnoxious and malodorous nature of algae in water and water supplies has been recognized for a long time; and their deteriorating effect of receiving water quality has also been recognized for a long time (8). Yet, the contribution to these receiving water quality problems by algae-laden lagoon effluents has gone substantially unrecognized.

Lagoons often optimize a severe overgrowth of

FIGURE 1. A GREEN SWATCH OF ALGAE, THE CANCER OF WATER POLLUTION RIMS THE ROCKY BEACH OF A DYING LAKE(7).

blue-green algae. Oswald and Gloueke (6) have reported algae accumulations in lagoons to constitute about 25,000 pounds of dry algae per acre. These algae mats may enter the effluent stream and cause severe deterioration of the receiving water quality; or they may decay, causing obnoxious "pig-pen" odors. In addition, some blue-green algal extracellular products are known toxins (9,10,11) and algae blooms and subsequent decay are known to impart taste and odors to receiving waters subsequently serving as a water supply (8).

1.1 PROBLEM

During the past three decades of accelerated acceptance and installation of oxidation ponds, much work has been done in two general areas:

1. Algae production and classification, as related to the natural purification process operative in oxidation ponds (12,13,14,15).
2. Development of design criteria for organic and hydraulic loading of oxidation ponds (16,17,18).

Lacking in the literature has been a comprehensive evaluation of lagoon effluent quality as evidenced by excessive effluent BOD, suspended solids (SS), and organic contaminants. Also lacking has been a comprehensive evaluation of the numerous periodic oxidation pond aesthetic failures discussed in the introduction. Lagoon performance has, heretofore, never been evaluated from the position of water quality criteria; nor has a comprehensive effort been made to evaluate the physical, chemical and biological limiting factors which define lagoon performance.

Because oxidation ponds constitute the most numerous treatment process used in many areas of the United States and because of the many periodic problems associated with oxidation ponds, it is important that lagoon performance be evaluated on an effluent and aesthetic quality basis.

1.2 PURPOSE

That oxidation ponds do not substantially contribute to the enhancement of receiving water quality has never been demonstrated; nor have physical, chemical or biological limiting factors been implicated with oxidation pond effluent and aesthetic quality failures.

The purpose of this study, then, was to investigate and report if:

1. The effluent BOD and suspended solids from raw sewage oxidation ponds would or would not meet effluent quality standards equivalent to secondary treatment and necessary to restore and maintain receiving water quality.
2. Tertiary lagooning is or is not an adequate reliable method to accomplish further removal of BOD, suspended solids, nitrogen and phosphorus compounds or indicator organisms from secondary treated wastewaters.
3. Oxidation ponds experience periodic aesthetic failures which, due to current design and operation practices, are substantially uncontrollable.

1.3 SCOPE

Utilizing Federal receiving water quality criteria (20) and Illinois Department of Public Health effluent standards (19), several hundred validated cases of raw sewage, facultative, aerated facultative, aerobic complete-mix and tertiary oxidation ponds were evaluated for aesthetic and effluent quality failures. Lagoons were selected to be representative of a variety of geographic areas and climatic conditions. Both raw sewage and tertiary lagoons were evaluated from the far west, wouthwest, midwest, upper midwest and southeast. Lagoons ranged in surface area from 0.05 acres to 105 acres. Parameters of concern were BOD, suspended solids (SS), COD, nitrogen, phoshporus, indicator organisms and aesthetic values.

1.4 HISTORY, EXTENT AND USE OF OXIDATION PONDS

It is uncertain when and where ponds were first used in the United States. During the mid-twenties, cities in California, Texas and North Dakota used lagoons as a means of treating municipal sewage (21). Oswald (22) reports that a small ponding installation at Bitterwater, San Benito County, California, was constructed about 1916 to receive oil pumping station wastes, waste boiler feed water and domestic sewage.

In 1924, Santa Rosa, California uncovered gravel pits which the city council thought could be used as natural filters prior to discharging city sewage into the then highly-polluted Santa Rosa Creek (23). The gravel pit soon became clogged and the pit filled to a depth of about three feet and thereafter operated as an oxidation pond.

In 1925, a lagoon irrigation arrangement was set up by V. M. Ehlers for the City of Abilene (21). Based upon this success, an experimental pond was built at Texas A & M College in 1929 (21).

The first lagoon of record in North Dakota was in the town of Fessenden and was placed in operation during 1928 (24). This community had no nearby stream to handle the effluent from the new sewage system completed during that year; so it was decided to discharge the effluent into a pothole about a mile west of town (25).

In the early stages of World War II, rapid and unpredictable population increases at military installations as well as at neighboring municipalities brought about the construction of numerous oxidation ponds to cope with the resultant overloads upon existing facilities. The first deliberately designed stabilization pond based on engineering concepts of treating raw sewage was first placed in operation in 1949 in Maddock, North Dakota (25). This installation aroused considerable enthusiasm on the part of the North Dakota State Health Department and shortly thereafter throughout the midwest and southwest. The accelerated extent and use of oxidation ponds had begun.

The accelerated use of oxidation ponds, as typified by the State of Missouri, may be seen in *Figure 2*. The numbers of lagoons serving municipalities, resorts, motels schools, subdivisions, slaughter houses, poultry processing plants, milk plants and automatic laundries were reported by Smith (27,28). The accelerated increase in the construction of oxidation ponds may be seen clearly in *Figure 2*. As of June 30, 1966, Smith (28) reports 1,375 operating permits for waste treatment works in Missouri, of which 908 are oxidation ponds. Today, in many parts of America, oxidation ponds represent the most numerous process for treating domestic and industrial wastes.

II EFFLUENT AND AESTHETIC QUALITY CRITERIA

2.1 FEDERAL RECEIVING WATER QUALITY CRITERIA

The 1961 Federal Water Pollution Control Act (29), as amended by the 1965 Water Quality Act (30), authorized the states and the Federal Government to establish water quality standards for interstate (including coastal) waters

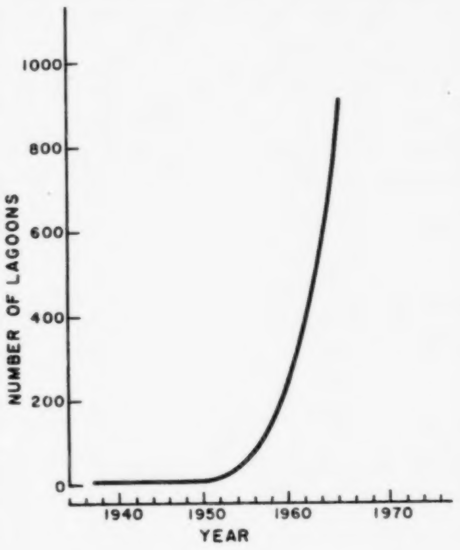

FIGURE 2. ACCELERATED DEPLOYMENT OF OXIDATION PONDS IN MISSOURI (27) (28).

by June 30, 1967. The water quality standards submitted by the states were subject to review by the Department of the Interior and, if found consistent with Paragraph 3 of Section 10 of the Water Quality Act of 1965, would be approved as Federal standards by the Secretary of the Interior. Paragraph 3, Section 10 is as follows:

> Standards of quality pursuant to this subsection shall be such as to protect the public health or welfare, enhance the quality of water and serve the purposes of this Act. In establishing such standards the Secretary, the Hearing Board or the appropriate state authority shall take into consideration their use and value for public water supplies, propagation of fish and wild life, recreational purposes and agricultural, industrial and other legitimate uses.

On February 27, 1967, the Secretary of the Interior established the first National Technical Advisory Committee on Water Quality Criteria to the Federal Water Pollution Administration (20). The Committee's prinicpal function was to collect into one volume a basic foundation of water quality criteria. The Committee was subdivided to develop criteria for five general areas of water use: 1) Recreation and Aesthetics, 2) Public Water Supplies, 3) Fish, Other Aquatic Life and Wild Life, 4) Agriculture and 5) Industry.

These recommended criteria for the various legitimate water uses can serve as effluent quality criteria. That is, the legally established water uses of a receiving water concomitantly establishes maximum effluent quality criteria necessary to maintain those legitimate uses. Based, then upon the legitimate water uses; the physical, chemical and biological nature of the waste-water; and the available dilution and assimilative capacity of the receiving waters; the degree of treatment required by waste-waters to maintain legitimate uses of the receiving waters may be established.

The standards adopted by the States would include water use classifications, criteria necessary to support these uses, enforcement and plan for implementation which would include acceptable methods of treatment. Selection of treatment methods would be dependent upon the process' ability to produce an effluent that would meet criteria necessary to maintain established water uses and enhance receiving water quality (20).

Prior to treatment method selection that would produce an effluent to meet criteria necessary to maintain established water uses and enhance receiving water quality, it is necessary to underatand the intent of the Water Quality Criteria recommended by the National Technical Advisory Committee (20).

2.1.1 Recreation and Aesthetic Criteria

It was the intent of the Advisory Committee (20) to provide aesthetic criteria to protect surface waters from floating debris, oil and scum, objectionable deposits and substances producing objectionable color, odor, taste or turbidity. Based upon this intent, a treatment process which periodically produced: a highly-colored effluent; algae blooms that impart tastes and odors; obnoxious hydrogen sulfide and septic sewage odors; objectionable floating algae mats or shore deposits which decay giving off a "pig-pen" odor; may not be [illegible] to maintain or enhance receiving water quality and may not be suitable for discharge to receiving waters

Regarding recreational uses, it was the intent of the National Technical Advisory Committee to provide surface waters suitable for human use in recreation activities not involving significant risks of ingestion. Surface waters should be of such quality as to support fishes, water fowl and other forms of aquatic life. Species available for human harvest should be fit for human consumption.

2.1.2 Public Water Supply Criteria

Because treatment processes exist which can, at a price, convert almost any water including sea water and grossly polluted fresh water into a potable product, it is necessary to consider the type of treatment in any discussion of raw water quality criteria for public water supplies. The National Technical Advisory Subcommittee on Public Water Supplies (20) adopted as the considered treatment the most common processes in use in this country. This may include coagulation, sedimentation, san filtration and disinfection.

It was the intent of the Advisory Committee that raw water should be virtually absent of objectionable odors and tastes. A limit of 75 color units was recommended by the Committee to permit the defined treatment plant to produce water meeting Drinking Water Standards (31) with moderate dosages of coagulant chemicals. The Committee provided that turbidity must be readily removable and frequently changing turbidity is damaging to public water supply.

The intent of the Public Water Supply Subcommittee was to protect raw water supplies from odors, tastes and color. Oxidation pond effluents, however, periodically impart tastes, odors and color to receiving waters. Color is usually due to green or blue-green algae which grow and remain in suspension in the lagoon effluent. Tastes and odors are imparted to receiving waters by algae extracellular products or may result from the decay of floating algae mats or sludge deposits.

2.1.3 Fish, Other Aquatic Life and Wild Life Criteria

The task of the National Technical Advisory SubCommittee on Water Quality Criteria for Fish, Other Aquatic Life and Wild Life (20) was to describe, insofar as possible, the environmental requirements of aquatic life and wildlife and the environmental concentrations of potential toxicants that are not harmful under long-term exposure.

Substances in suspension and solution, whether solid, liquid or gas, largely determine the quality of the water. Aquatic organisms are affected not only directly by those materials, but also indirectly through their effects on other forms of aquatic life which comprise their food, competitors and predators. Hence, the determination of water quality requirements for aquatic life is a very involved task.

In approaching the problem of protecting aquatic and wild life resources, it was the intent of the Subcommittee (20) to realize and emphasize that:

1. Certain natural complexes of dissolved materials to which aquatic organisms have become adapted are favorable; whereas, other concentrations or compositions may not be favorable.
2. Altering the amount of substances normally found in the environment can be harmful.
3. Toxicity is a quantitative term; any material becomes toxic when its concentration exceeds certain levels. It is essential that requirements be maintained through periods of natural fluctuations as low water, maximum discharge and minimum dissolved oxygen.

The intent of the Aquatic Life and Wild Life Subcommittee was to protect receiving waters from unnatural fluctuations of dissolved solids, turbidity and oxygen-depleting substances. Oxidation pond effluents, however, periodically impart turbidity to receiving waters. Turbidity is usually due to algae and other microorganisms which grow and remain in suspension in the lagoon effluent. Unoxidized organics in lagoon effluents, as short-circuited raw wastewater and algae or other microbial cellular material, will deplete the oxygen reserves of the receiving water. In addition, the discharge of blue-green algae in lagoon effluents which do not participate in the food chain may actually be toxic to aquatic life. It is apparent that under certain circumstances lagoon effluents may not enhance or maintain receiving water quality sufficient to protect fish, other aqautic life and wild life.

2.1.4 Agricultural Use Criteria

The Subcommittee on Water Quality for Agricultural Uses (20) was concerned with water used on individual farmsteads for livestock and for irrigation of crops.

The objectionable foreign matter in farmstead water which is influenced by the method of wastewater treatment can be divided into two groups:

1. Suspended matter. This includes biological solids as algae and other microorganisms and inorganic solids.
2. Materials causing tastes, odor and color. These impurities normally occur as the result of one of the following: algae, dissolved organic matter, dissolved organic gases, hydrogen sulfide, earthy constituents, phenols or other wastes.

Algae are the most frequent causes of taste and odors in farm water supply systems (20). Some types of algae serve as the basis of the food chain. Other types of algae, notably the blue–green type, are toxic and can cause death of both aquatic species and livestock (32,33,34,35).

The intent of the Agricultural Use Subcommittee was to protect receiving waters suitable for farmstead, livestock and irrigation uses from suspended biological solids and materials causing tastes, odors and color. Oxidation pond effluents, however, periodically impart tastes, odors and color to receiving waters. Color is usually due to green or blue–green algae which drow and remain in suspension in the lagoon effluent. Tastes and odors are imparted to receiving waters by algae extracellular products, or may result from the decay of floating algae mats or sludge deposits. Toxins released by certain blue-green algae, which frequent the lagoon environment, have been reported to cause death of both aquatic species and livestock. It is apparent that, under certain circumstances, oxidation pond effluents further degrade the quality of receiving waters.

2.1.5 Industrial Use Criteria

The National Technical Advisory Subcommittee for Water Quality Requirements for Industrial Water Supplies (20) has reached the following conclusions regarding the water quality requirements for industrial supplies'

1. The quality characteristics of the water supply for an established industry at a given site, if allowed to deteriorate from the range usually experienced for those characteristics of significance to that industry, can sause an undersirable increase in the cost for treatment and deterioration of product quality for some industries.
2. The industrial water quality requirements are generally well established for each industrial use.
3. Process water quality may vary considerably between plants of the same industry depending on the technological age of the design, the quality of the available supply and cost of treatment.

The intent of the Industrial Use Subcommittee was to protect raw water supplies from wide fluctions in water quality which could deteriorate product quality or increase the cost of treatment. Oxidation pond effluents, as discussed above, periodically impart taste, odors and color to receiving waters which could significantly deteriorate product quality or greatly increase the cost of water treatment.

2.2. STATE EFFLUENT QUALITY STANDARDS

The cornerstone of America's clean water program is the provision of the 1965 Water Quality Act (30). This Act called for all states to establish water quality criteria (similar to the above Federal guidelines criteria) for their interstate and coastal waters and required states to make crucial decisions concerning the uses of their water resources, the quality of water to support these uses and specific plans for achieving such levels of quality.

In compliance with the Water Quality Act of 1965, the State of Illinois Sanitary Water Board, Division of Sanitary Engineering, Illinois Department of Public Health, has designated acceptable treatment methods and established effluent standards necessary to resore and maintain receiving water quality (19). These standards are presented in *Table 1*. To the extent that all states have receiving waters with similar legitimate uses, the Illinois Standards can be used as an approved example of those to be adopted by other states (20,30).

The Illinois Act (19) defines sewage as "the water–carried human or animal wastes from residences, buildings, industrial establishments or other places." Policy, under the Act, is to provide that no waste be discharged into any waters of the State without first being given the degree of treatment necessary to prevent the pollution of such waters. Because of the diverse nature of waterborne wastes, each proposed installation will be considered on its own merits with regard to the nature and strength of the waste and the use and capacity of the receiving water of the State. However, certain basic standards have been established.

Based upon the receiving water quality requirements in Illinois, the State Sanitary Water Board (19) established the permissive treatment methods and effluent quality requirements summarized in *Table 1*. It may be seen that all facilities for treatment of deoxygenating waste shall provide at least secondary biological treatment or advanced waste treatment adequate to reduce the organic pollution load of the treatment works effluent at the final treatment structure to a maximum of 40 mg/l of 5–day BOD and 45 mg/l of suspended solids, or less, as required by the specific receiving water. For example, it may be seen in *Table 1* that for a receiving water providing 2-to-1 dilution, the effluent BOD and suspended solids are required to be less than 20 mg/l and 24 mg/l repsectively. For receiving waters providing less than 1-to-1 dilution, the effluent BOD and suspended solids are required to be less than 4 mg/l and 5 mg/l respectively.

The intent of these standards is to enhance and maintain receiving water quality by requiring all facilities treating organic wastes to provide:

1. Substantially complete removal of settleable solids.
2. Removal of floating debris, surface mats, scum, oil, grease or sludges.
3. Removal of color, odor or turbidity to below obvious levels.
4. Effluent disinfection with up to 1 mg/l of chlorine residual in the final effluent discharges to the to the receiving waters to reduce coliform to 5000/100 ml or less, or reduce fecal coliform to 400/100 ml in primary contact recreation areas.

Based on the effluent standards, presented in Table 1, a treatment process which periodically produces:

1. Effluent BOD values in excess of acceptable standards;
2. Effluent suspended solids valves in excess of acceptable standards;
3. Algae blooms that impart tastes and odors;
4. Algae mats and scum which decay and give off "pig-pen" odors and form sludge deposits; and
5. A highly–colored effluent;

may not be suitable for discharge to receiving waters. These periodic failures will be shown to be charateristic of oxidation ponds.

TABLE 1

Effluent Quality Requirements and
Acceptable Treatment Methods[*]

Stream Requirements	Type of Treatment	Type of Facilities	Effluent BOD_5, mg/l		Effluent SS, mg/l
			Conc., mg/l	% Removal	
Miss. River Ohio River Lake Mich.	Secondary	Biological	40	80	45
Ill. River Des Plains Riv.	Secondary	Trickling Filter	30	85	35
2 to 1 Dilution	Secondary	Secondary plus[**] Supplemental	20	90	25
1 to 1 Dilution	Tertiary	Secondary plus[**] Supplemental	10	95	13
Less than 1 to 1 Dilution	Advanced	Series Units	4	98	5

[*] State of Illinois Sanitary Water Board, Division of Sanitary Engineering, Illinois Department of Public Health (19).

[**] Activated sludge or trickling filter followed by supplemental treatment.

Thus, from the standards shown in Table 1, any treatment process may be evaluated for its ability to achieve a level of effluent quality; of the inability of a process to achieve a designated effluent quality may be assigned a degree of failure. Standards for failure, however, should agree with the objectives of the process. That is, standards gauging failure of secondary treatment processes, as raw sewage lagoons, should not be applicable to tertiary treatment processes, as tertiary lagoons.

Based on the above considerations and the effluent quality standards presented in Table 2, degrees of failure of raw sewage oxidation ponds were developed. These results, shown in Table 2, are to be interpreted as follows:

1. 1st Degree Failure: 20 mg/l BOD_5 25 mg/l SS An effluent containing 20 mg/l BOD_5 and 25 mg/l SS represents the maximum permissible effluent concentration allowed by the State of Illinois for discharge to receiving waters providing 2 to 1 dilution.
2. 2nd Degree Failure: 30 mg/l BOD_5, 35 mg/l SS An effluent containing 30 mg/l BOD_5 and 35 mg/l SS corresponds to the maximum permissible effluent concentrations allowed by the State of Illinois for discharge to receiving waters with water use criteria similar to the MIssissippi River, Ohio River, or Lake Michigan. Effluents containing greater than 40 mg/l BOD_5 and 45 mg/l SS are inadequately treated and are considered by the State of Illinois to be unacceptable for discharge to any receiving waters.
3. 3rd Degree Failure: 40 mg/l BOD_5, 45 mg/l SS An effluent containing 40 mg/l BOD_5 and 45 mg/l SS corresponds to the maximum permissible effluent concentrations allowed by the State of Illinois for discharge to receiving waters with use criteria similar to the Mississippi River, Ohio River, or Lake Michigan. Effluents containing greater than 45 mg/l BOD_5 and 45 mg/l SS are inadequately treated and are considered by the State of Illinois to be unacceptable for discharge to any receiving waters.

Tertiary oxidation ponds, as the name infers, are considered tertiary or advanced waste treatment processes. As such, Illinois State effluent standards, shown in Table 1 for tertiary and advanced waste treatment processes, may be considered effluent standards by which tertiary oxidation pond effluents may be evaluated. These results shown in Table 3 are to be interpreted as follows:

1. 1st Degree of Failure: 4 mg/l BOD_5 5 mg/l SS. An effluent containing 4 mg/l BOD_5 and 5 mg/l SS represents the maximum permissible effluent concentrations allowed by the State of Illinois for discharge to receiving waters providing less than 1 to 1 dilution.
2. 2nd Degree Failure: 10 mg/l BOD_5 13 mg/l SS An effluent containing 10 mg/l BOD_5 and 13 mg/l SS represents the maximum permissible concentrations allowed by the State of Illinois for discharge to receiving waters providing 1 to 1 dilution.
3. 3rd Degree of Failure: 20 mg/l BOD_5 25 mg/l SS An effluent containing 20 mg/l BOD_5 and 25 mg/l SS represents the maximum permissible concentrations allowed by the State of Illinois for discharge to receiving waters provoding 2 to 1

dilution.

The results of Table 2 and Table 3, which define oxidation pond failures, were used to evaluate the results of oxidation pond effluent quality characteristics.

In the following section, periodic oxidation pond effluent and aesthetic quality failures were reported in relationship to Federal Receiving Water Quality (20) and Illinois Effluent Standards (19). It may be seen from these documented reports that oxidation ponds may not be able to achieve effluent quality standards necessary to enhance or maintain receiving water quality and may not be considered equivalent secondary treatment.

TABLE 2

DEGREE OF FAILURE OF
RAW SEWAGE OXIDATION PONDS
BASED ON EFFLUENT QUALITY STANDARDS
FOR BIOCHEMICAL OXYGEN DEMAND (BOD) AND SUSPENDED SOLIDS (SS)

DEGREE OF FAILURE	EFFLUENT BOD_5 CONC. mg/l	EFFLUENT BOD_5 % REMOVAL	EFFLUENT SS, mg/l
1st	20	90	25
2nd	30	85	35
3rd	40	80	45

TABLE 3

DEGREE OF FAILURE OF
TERTIARY OXIDATION PONDS
BASED ON EFFLUENT QUALITY STANDARDS
FOR BIOCHEMICAL OXYGEN DEMAND (BOD) AND SUSPENDED SOLIDS

DEGREE OF FAILURE	EFFLUENT BOD_5 CONC. mg/l	EFFLUENT BOD_5 % REMOVAL	EFFLUENT SS, mg/l
1st	4	98	5
2nd	10	95	13
3rd	20	90	25

III RESULTS & DISCUSSION

3.1 EFFLUENT BOD_5 FAILURES – RAW SEWAGE OXIDATION PONDS

The degree to which 21 cases of facultative raw sewage oxidation ponds exceeded effluent BOD standards may be seen in *Figure 3.*. It may be noted that 16 of the 21 cases (76 per-cent) exceeded a third degree failure. By the Illinois State effluent standards presented in *Table 1*, these 16 effluents would not be acceptable for discharge into any receiving waters. The remaining 5 cases (all 2nd degree BOD failures) produced effluents of marginal quality, acceptable for discharge only to the Mississippi and Ohio Rivers or Lake Michigan. In no case was a raw sewage facultative lagoon effluent produced that was acceptable for receiving waters providing 2 to 1 dilution.

The degree to which aerated facultative lagoons and complete-mix aerobic lagoons met effluent BOD standards has been plotted in *Figure 4*, along with the range in effluent BOD from raw sewage facultative lagoons. Facultative raw sewage lagoon effluent BOD values ranged from 33 to 135 mg/l, with a median of 50 mg/l. Aerated facultative lagoon effluent BOD values ranged from 25 to 89 mg/l, with a median of 43 mg/l. Complete-mix aerobic lagoon effluent BOD values ranged from 107 to 299 mg/l, with a median of 180 mg/l. It may be noted that 76 percent of the raw sewage facultative lagoons, 67% of the aerated facultative lagoons, and 100% of the complete complete-mix aerobic lagoons exceed a 3rd degree failure (based on BOD): and therefore are unacceptable for discharge to any Illinois receiving waters.

The degree to which these lagoon effluent BOD values exceed the Illinois effluent standards may also be seen in *Figure 4*. It may be seen from the ranges and median effluent BOD values that oxidation ponds are not suited to producing an effluent quality acceptable for discharge into Illinois receiving waters.

Although the lagoon effluent BOD quality presented in *Figures 3 and 4* in most cases greatly exceeded the effluent BOD standards presented in *Table 2*, it should be noted that the total organic pollution content of the effluents was even greater than that measured by the BOD evaluations. That is, the 5-day BOD for domestic sewage is normally about 70 to 80% of the ultimate BOD (36). However, this ratio is considerably less for algae-laden lagoon effluents (37,38,39). The importance of utilizing supplmental evaluations, as suspended solids, volatile suspended solids, COD, and organic carbon determinations on lagoon effluents will be discussed in the following section.

3.2 EFFLUENT SUSPENDED SOLIDS (SS) FAILURES – RAW SEWAGE OXIDATION PONDS

The degrees to which 15 cases of facultative raw sewage oxidation ponds met suspended solids (SS) standards were presented in*Table 4*. The degrees to which these lagoon effluents exceeded the effluent standards may be seen in *Figure 5*. It may be noted that 100% of these cases exceeded a 3rd degree failure. By the Illinois effluent standards presented in *Table 2*, 100% of these effluents would not be acceptable for discharge into any receiving waters. In no case was a raw sewage facultative lagoon able to meet effluent suspended solids standards. In fact, in 9 of the 15 cases, the effluent suspended solids exceeded the maximum permissible by 100 to 200 fold.

The degree to which aerated facultative lagoons and complete-mix aerobic lagoons met effluent SS standards was presented in *Tables 5 and 6*. The range in effluent SS values from these types of lagoons has been plotted in *Figure 6*, along eith the range in effluent suspended solids from raw sewage facultative lagoons. Facultative raw sewage lagoon effluent BOD values ranged from 62 to 7,367 mg/l, with a median value of 2,020 mg/l. Aerated facultative lagoon effluent SS values ranged from 20 to 140 mg/l, with a median of 75 mg/l. Complete-mix aerobic lagoon effluent SS values ranged from 166 to 434 mg/l, with a median value of 277 mg/l. It may be noted that 100% of the raw sewage facultative lagoons, 77% of the aerated facultative lagoons, and 100% of the complete-mix aerobic lagoons exceed a 3rd degree SS failure; and, therefore, are unacceptable for discharge to any Illinois receiving waters.

The degree to which these lagoon effluents exceed the Illinois effluent standards may also be seen in *Figure 6*. It may be seen from the ranges and median effluent SS values that oxidation ponds are not suited to producing an effluent quality acceptable for discharge into Illinois receiving waters.

3.3 EFFLUENT FAILURES – TERTIARY OXIDATION PONDS

Conventional treatment may not prepare wastewater adequately for discharge to particular receiving waters. For this reason, it may be desirable to remove greater amounts of polluting elements still present in the effluents of conventional secondary processes. Tertiary facilities, then, are designed to accomplish further removal of BOD, suspended solids, nitrogen and phosphorus compounds and indicator organisms.

3.3.1 Effluent BOD_5 Failures - Tertiary Lagoons

The degrees to which 24 cases of tertiary oxidation ponds met or exceeded effluent BOD standards may be seen in *Figure 7*.

Of the 15 cases which document the total effluent BOD, 12 of these cases (80%) exceeded a 3rd degree failure. These 12 effluents would not be acceptable for discharge to receiving waters providing less that 2 to 1 dilution.

Cases 7, 9, 11, 13, 15, 17 and 19 are BOD values of effluent filtrates. The BOD of these 7 cases ranged from 6 to 9 mg/l and are, from this point of view, suitable for discharge to Illinois receiving waters providing a minimum 1 to 1 dilution. Of these same 7 cases, the BOD of the unfiltered effluents (cases 6, 8, 10, 12, 14, 16 and 18) ranged between 19 and 24 mg/l. The limited acceptability of these filtered effluents demonstrates the importance of affecting solids separation prior to discharge of a tertiary oxidation pond effluent.

3.3.2 Effluent Suspended Solids Failures - Tertiary Lagoons

The degrees to which 23 cases of tertiary oxidation ponds met or exceeded suspended solids standards may be seen in *Figure 8*. It may be noted that 15 out of 16 cases reporting total effluent suspended solids exceeded a 3rd degree failure. That is, by the Illinois effluent standards

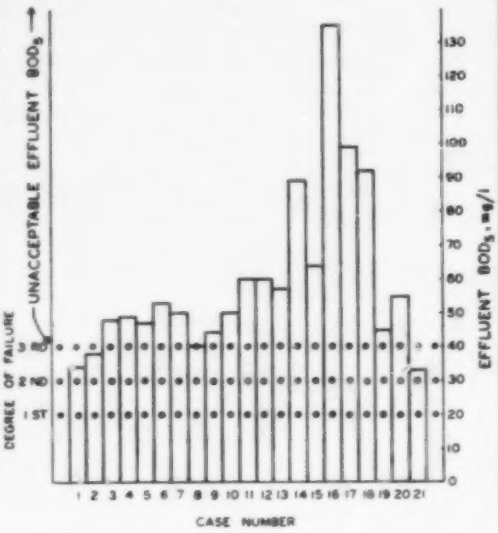

FIGURE 3 THE DEGREE OF FAILURE OF 21 CASES OF RAW SEWAGE FACULTATIVE OXIDATION PONDS BASED ON EFFLUENT BOD_5.

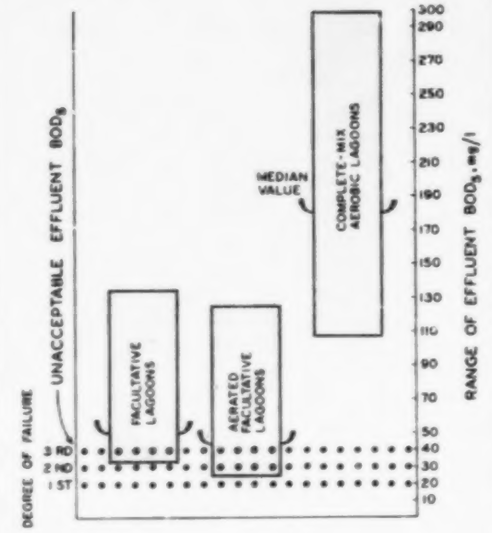

FIGURE 4 THE DEGREE OF FAILURE AND THE RANGE OF EFFLUENT BOD_5 FROM FACULTATIVE, AERATED FACULTATIVE AND COMPLETE-MIX AEROBIC RAW SEWAGE OXIDATION PONDS.

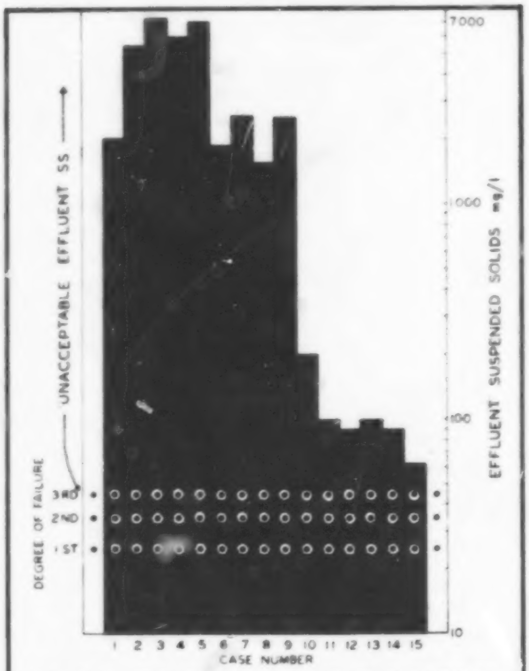

FIGURE 5 THE DEGREE OF FAILURE OF 15 CASES OF RAW SEWAGE FACULTATIVE OXIDATION PONDS BASED ON EFFLUENT SUSPENDED SOLIDS.

TABLE 4

DEGREE OF FAILURE OF
15 CASES OF FACULTATIVE OXIDATION PONDS*
(BASED ON EFFLUENT SUSPENDED SOLIDS, SS)

CASE	LOCATION	EFFLUENT SS		FAILURE	REFERENCE
		AV. CONC., mg/l	% REMOVAL		
	Fayette, Mo.				
1	Pond 1	2970**	[illegible]***	3rd.	(47)
2	Pond 2	5540**	[illegible]***	3rd.	(47)
3	Pond 3	7367**	[illegible]***	3rd.	(47)
4	Pond 4	6009**	[illegible]***	3rd.	(47)
5	Pond 5	7111**	[illegible]***	3rd.	(47)
	Fayette, Mo.				
6	Pond 1	1951**	[illegible]***	3rd.	(52)
7	Pond 3	2579**	[illegible]***	3rd.	(52)
8	Pond 4	1542**	[illegible]***	3rd.	(52)
9	Pond 5	2582**	[illegible]***	3rd.	(52)
	[illegible], Mo.				
10	Pond 1	200	64.	3rd.	[illegible]
	Miss. State U.				
11	Pond 1	100	57	3rd.	(51)
12	Pond 1	90****	50	3rd.	(51)
13	Pond 2	100	57	3rd.	(51)
14	Pond 2	90****	50	3rd.	(51)
	Red Bridge, Mo.				
15	No. 2 Lagoon	62	52	3rd.	(59)

* Facultative oxidation ponds receive raw sewage; no supplemental aeration.

** Values reported as PPM/Vol were converted to mg/l based on a specific gravity of algae to be one, as recommended in Standard Methods (49).

*** Minus sign indicates effluent concentration exceeds influent concentration.

**** Volatile suspended solids.

TABLE 5

DEGREE OF FAILURE OF
16 CASES OF AERATED FACULTATIVE OXIDATION PONDS*
(Based on Effluent Suspended Solids, SS)

CASE	LOCATION	EFFLUENT SS		FAILURE	REFERENCE
		AV.CONC.mg/l	% REMOVAL		
	Raytown, Mo.				
1	1st. Stage	130	35	3rd.	(53)
2	2nd Stage	78	40	3rd.	(53)
	Lee's Summit, Mo.				
3	Little Cedar	88	50	3rd.	(53)
	Red Bridge, Mo.				
4	No. 2 Lagoon	88	(-) 5**	3rd.	(59)
	Ontario, Canada				
5	Skyline	75	77	3rd.	(60)
6	Skyline	51	87	3rd.	(60)
7	Skyline	37	91	2nd.	(60)
8	Skyline	82	n.a.***	3rd.	(60)
9	Skyline	43****	n.a.***	2nd.	(60)
10	Skyline	40	n.a.***	2nd.	(60)
11	Skyline	25****	n.a.***	None	(60)
12	Skyline	63	87	3rd.	(60)
13	Grimsby	26	80	1st.	(60)
14	Grimsby	20	62	None	(60)
	Saskatchewan, Ca.				
15	Regina Pond 1	140	n.a.***	3rd.	(61)
16	Regina Pond 2	98	n.a.***	3rd.	(61)

* Aerated facultative lagoon receives raw sewage and supplemental aeration.
** Minus sign indicates effluent concentration exceeds the influent concentration.
*** n.a. not available.
**** Volatile suspended solids.

TABLE 6

DEGREE OF FAILURE OF
13 CASES OF AEROBIC COMPLETE MIX OXIDATION PONDS*
(Based on effluent Suspended Solids, SS)

CASE	LOCATION	EFFLUENT SS		FAILURE	REFERENCE
		AV.CONC.mg/l	% REMOVAL		
	Ontario Canada				
1	Windsor 8/65	434	(-) 85***	3rd.	(60)
2	Windsor 8/65	215**	(-) 27***	3rd.	(60)
3	Windsor 10/65	258	(-) 27***	3rd.	(60)
4	Windsor 10/65	125**	(-) 14***	3rd.	(60)
5	Windsor 4/66	340	(-)106***	3rd.	(60)
6	Windsor 1966-1969	277	(-) 55***	3rd.	(60)
7	Windsor 1966-1969	185**	(-) 62*** (-)	3rd.	(60)
8	Durham 5/68	166	(-) 8.5***	3rd.	(60)
9	Durham 5/68	130**	(-) 10***	3rd.	(60)
10	Durham 6/68	180	n.a.****	3rd.	(60)
11	Durham 6/68	135**	(-) 23***	3rd.	(60)
12	Durham 1967-1968	294	(-) 79***	3rd.	(60)
13	Durham 1967-1968	240**	(-)100***	3rd.	(60)

* Raw sewage influent; complete solids suspension due to artifically induced turbulence.
** Volatile suspended solids.
*** Minus sign indicates effluent concentration exceeds influent concentration.
**** n.a. not available.

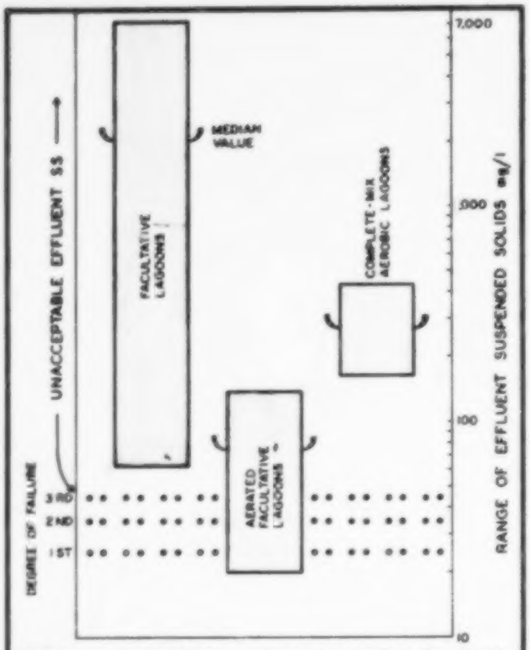

FIGURE 6 THE DEGREE OF FAILURE AND THE RANGE OF EFFLUENT SUSPENDED SOLIDS FROM FACULTATIVE, AERATED FACULTATIVE AND COMPLETE-MIX AEROBIC RAW SEWAGE OXIDATION PONDS.

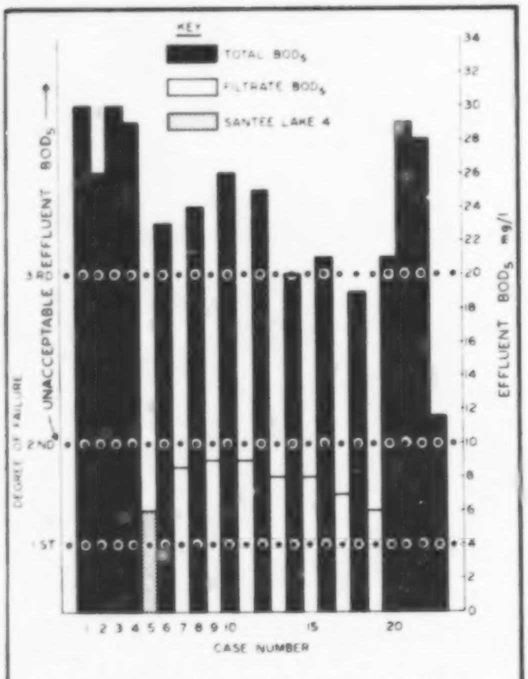

FIGURE 7 THE DEGREE OF FAILURE OF 24 CASES OF TERTIARY LAGOONS BASED ON EFFLUENT BOD5.

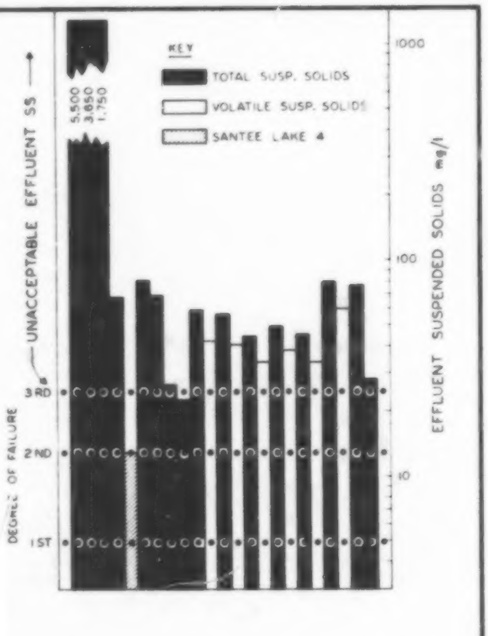

FIGURE 8 THE DEGREE OF FAILURE OF 23 CASES OF TERTIARY OXIDATION PONDS BASED OF EFFLUENT SUSPENDED SOLIDS.

presented in *Table 3*, 94% of these tertiray oxidation ponds were unable to produce an effluent quality consistent with tertiary standards necessary to enhance receiving water quality.

Cases 11, 13, 15, 17, 19 and 21 represent volatile suspended solids determinations on tertiary oxidation pond effluents. These volatile suspended solids results are significant because they measure the amount of organic matter present by a combustion procedure in which organic matter is converted to carbon dioxide and water. When conducted in accordance with Standard Methods (49), the weight loss incurred is a reasonably accurate measure of organic matter. The volatile suspended solids of these six cases ranged from 34 to 60 mg/l. All six cases (100%) exceeded a 3rd degree tertiary failure. It is clear that the volatile portion of the suspended solids by itself exceeds the tertiary standards of the State of Illinois.

Many investigators (50,51) have failed to report oxidation pond effluent suspended solids. However, Sawyer and McCarty (36) have indicated that suspended solids determinations are extremely valuable in the analysis of polluted waters. It is important to note that each milligram of algal suspended solids may exert 1.5 milligrams of oxygen demand (37) (38). In stream-pollution-control work, all suspended solids are considered to be settleable solids, as time is not a limiting factor. Sawyer and McCarty (36) indicate the deposition is expected to occur through biological and chemical flocculation; therefore, removal during treatment of organic suspended solids is considered fully as significant as BOD.

Microscopic examination of these lagoon effluents indicates the suspended solids to be predominately algae cells (47,52,53). These results indicate that the lagoon environment tends to optimize algae production. The obnoxious and malodorous nature of algae in eutrophic waters has long been recognized (8). Yet, the adverse effect of algae-laden lagoon effluents has gone substantially unrecongized. It is algae and their extra cellular products that cause accelerated deterioration of water quality (8). The lack of recognition of the adverse effect of these algae-laden effluents, their contribution to the problems of eutrophication and their subsequent recycle in nature has helped perpetuate a favorable opinion of oxidation ponds which in actuality is without evidence.

3.3.3 Degree of Phosphorus Removal - Tertiary Oxidation Ponds

The degrees of phosphorus removal in 31 cases of tertiary oxidation ponds were presented in *Table 5*. The degree to which these lagoon effluents accomplished removal of phosphorus may be seen in *Figure 9*. It may be noted that in 22 out of 30 cases the effluent phosphorus concentration exceeded the equivalent phosphorus form in the lagoon influent.

Orthophosphate removal ranged from (-) 175 to 52%, with a median removal of (-) 3%. These results indicate that orthophosphate removal was inconsistent and essentially nil. The removal of phoy-plus organic phosphorus ranged from (-)24 to 15%, with a median removal of (-)6%. These results indicate that removal of poly-plus organic phosphorus was nil. Considering the complexity of phosphorus chemistry, the removal of total phosphorus was considered the most representative determination of actual phosphorus removal. The removal of total phosphorus ranged from (-) 177 to 59%, with a median removal of (-)5%. These results do not approach the 80% reduction in phosphorus that is normally the objective of tertiary phosphorus removal (40,41). These results indicate that the tertiary oxidation pond is not a reliable nor effective method of removing phosphorus from domestic wastewater.

3.3.4 Degree of Nitrogen Reduction - Tertiary Oxidation Ponds

The degrees of total, ammonia and organic nitrogen removal in 19 cases of tertiary oxidation ponds were investigated. Removal of organic nitrogen ranged from (-)122% to 18%, with a median of (-)19%. These results indicate that tertiary lagoon effluents often contained more organic nitrogen than was present in the lagoon influent, a condition which is indicative of synthesis by primary producers. Ammonia nitrogen removals ranged from (-)21% to 17%, with a median removal of 47%. These results indicate that ammonia removal was inconsistent. The removal of total nitrogen ranged from (-)15% to 97%, with a median of 17%. It is clear from the total nitrogen removal results that nitrogen removal by tertiary lagooning is an ineffective and inconsistent process.

3.3.5 Degree of Coliform Reduction - Tertiary Oxidation Ponds

A major measure of tertiary pond performance is the effect on reducing the number of index organisms associated with fecal pollution. The degrees of coliform reduction in 12 cases of tertiary oxidation ponds were evaluated. Average reductions ranged from 48.9% to 96%. The results were highly erratic. In sewage treatment, the primary objective is to preserve and protect the public health. At times, the reduction of coliform organisms in tertiary lagoons is sufficient to meet receiving water criteria. Considering the success of chlorination, it is difficult to suggest or accept the premise that protection of the public health be placed on the erratic results of a tertiary oxidation pond (42).

3.4 THE GOAL OF NATURAL PURIFICATION VERSUS THE GOAL OF WASTE TREATMENT – RECYCLE VERSUS REMOVAL

In natural purification, energy–yielding life processes, as bacterial oxidation, combine oxygen with organic wastes $(CH_2O)_x$ producing carbon dioxide, water and energy:

$$CH_2O + O_2 \xrightarrow{bacteria} CO_2 + H_2O + energy$$

In Photosynthesis carried out by algae, energy is fixed as organic matter and oxygen is liberated, this:

$$CO_2 + 2H_2O + energy \xrightarrow{algae} CH_2O + O_2 + H_2O$$

These transformations are operative in natural purification and oxidation ponds. The elements contained in organic matter are repeatedly oxidized and synthesized, gaining energy through the combination of light energy. This cyclic process is known as the Law of Recycle.

A lagoon is a naturally complex biological ecosystem, comprised of reducers, producers and grazers. Influent putrescibel organics are digraded by bacteria which yield inorganic end products. These end products of the reducers are taken by the primary producers and synthesized to organic matter, which is passed up the food chain until death and recycle by reducers and primary producers. Thus, lagoons with their great surface area exposed to sunlight

TABLE 7

DEGREE OF PHOSPHORUS REMOVAL IN 31 CASES OF TERTIARY OXIDATION PONDS*

CASE	LOCATION	PHOS. FORM	EFFLUENT PHOSPHORUS-PO4 AV. CONC. MG/L	EFFLUENT PHOSPHORUS-PO4 AV.% REMOVAL	REFER-ENCE
	Santee, Calif.				
1	Tert. Pond [illegible]	Ortho-PO4	16.6	(-) 78**	(55)
2	Tert. Pond [illegible]	Ortho-PO4	13.2	52	(55)
3	Tert. Pond [illegible]	Total-PO4	27	56	(55)
4	Tert. Pond [illegible]	Ortho-PO4	52.6	(-)175**	(55)
5	Tert. Pond [illegible]	Tot.Filtrate	36.5	(-)179**	(55)
6	Tert. Pond [illegible]	Tot.Unfilt.	56.5	(-)150**	(55)
7	Tert. Pond [illegible]	Ortho-PO4	26.5	17	(55)
8	Tert. Pond [illegible]	[illegible]	17.[illegible]	[illegible]	(55)
9	Tert. Pond [illegible]	Ortho-PO4	24.0	8	(43)
10	Tert. Pond [illegible]	Total	30.0	(-) 20**	(43)
	Durham, No. Carolina				
11	Pond A1	Tot.Ortho-PO4	26.4	(-) 11**	(56)
12	Pond A[illegible]	Tot.Ortho-PO4	29.[illegible]	(-) [illegible]**	(56)
13	Pond B1	Tot.Ortho-PO4	26.4	(-) [illegible]**	(56)
14	Pond B[illegible]	Tot.Ortho-PO4	29.[illegible]	(-) 4**	(56)
15	Pond B[illegible]	Tot.Ortho-PO4	27.[illegible]	(-) 1**	(56)
	Durham, No. Carolina				
16	Pond 1	Ortho-PO4	21.[illegible]	(-) [illegible]**	(57)
17	Pond 1	Poly + Org	7.7	(-)10.[illegible]**	(57)
18	Pond 1	Total	[illegible].4	(-) 8.[illegible]**	(57)
19	Pond 2	Ortho-PO4	28.5	(-) [illegible]**	(57)
20	Pond 2	Poly + Org	[illegible]	(-) [illegible]**	(57)
21	Pond 2	Total	[illegible]	(-) [illegible]**	(57)
22	Pond 3	Ortho-PO4	28.[illegible]	(-) [illegible]**	(57)
23	Pond 3	Poly + Org	[illegible]	[illegible]	(57)
24	Pond 3	Total	[illegible]	[illegible]	(57)
25	Pond 4	Ortho-PO4	30.[illegible]	(-) [illegible]**	(57)
26	Pond 4	Poly + Org	[illegible].4	(-) [illegible]**	(57)
27	Pond 4	Total	[illegible]	(-) [illegible]**	(57)
28	Pond 5	Ortho-PO4	[illegible]	[illegible]	(57)
29	Pond 5	Poly + Org	[illegible]	(-) [illegible]**	(57)
30	Pond 5	Total	[illegible]	(-) [illegible]**	(57)
31	[illegible]	Total	2[illegible]	[illegible]	([illegible])

* [illegible]

** [illegible]

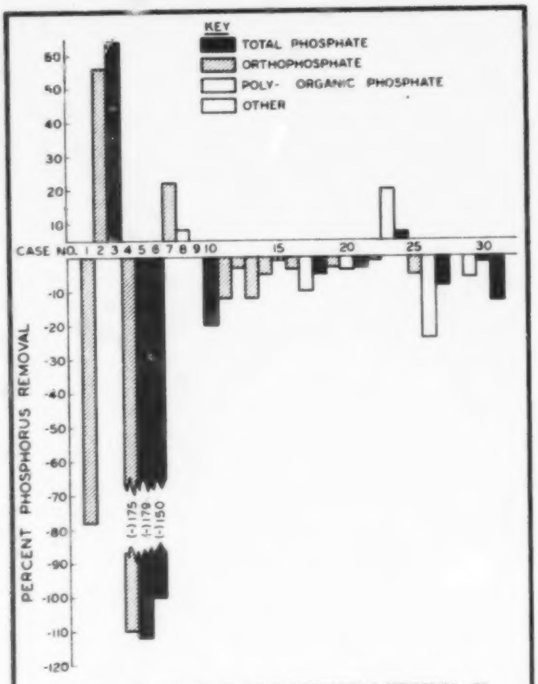

FIGURE 9 PERCENT OF PHOSPHORUS REMOVAL IN 30 CASES OF TERTIARY OXIDATION PONDS.

and their waters rich in nutrients serve as basins for the natural recycle of pollutants.

Oswald (13), McKinney (43), Gloyna (44), Parker (45), Drews (46) and others have verified this cyclic bacteria-algae symbiosis as the dominant treatment process operative in lagoons. For this reason, many investigators (5) have considered lagoons to be ideal forms of waste treatment.

It is true that lagoons more closely approach natural purification than any other treatment process; however, what has not been fully recognized is that the objectives of natural purification and waste treatment are different. As discussed above, the goal of natural purification is to recycle pollutants. The goal of waste treatment is to remove pollutants. Oxidation ponds, which emulate natural purification, recycle rather than remove pollutants. This is the fundamental reason why lagoons are unable to achieve acceptable effluent quality and why lagoons should not be considered equivalent to secondary treatment. Evidence of recycle of pollutants in tertiary oxidation ponds may be ascertained by analyzing influent-effluent balances of organic matter, suspended solids, nitrogen and phosphorus.

The percent suspended solids (SS) reduction in 23 cases of tertiary oxidation ponds is plotted in *Figure 10*. It may be noted that in 16 of the 23 cases (70%) the lagoon effluent suspended solids concentration exceeded the influent concentration by between 6 and 2,130%, with a median increase of 17%. In the remaining 7 cases, the removal of suspended solids ranged from 5 to 21%, with a median removal of 13%. Results of microscopic examination of these lagoon effluents indicate the suspended solids to be predominantly organic algae cells (47,48). It is evident from these results that the lagoon environment tends to optimize algae production. Algae-laden lagoon effluents carry pollutants (recycled) into the receiving waters. There, deposition, as indicated by Sawyer and McCarty (36), occurs through biological and chemical flocculation and the cyclic process. It is nature's propensity for recycle rather than removal that has led to highly eutrophic water masses. In order to enhance receiving water quality, waste treatment must affect removal of pollutants rather than recycle of pollutants prior to discharge. Because lagoons tend to recycle rather than remove pollutants, they cannot be expected to enhance receiving water quality nor can they be considered an equivalent secondary treatment process.

This fact is verified in *Figure 11*. The percent COD removal in 10 cases of tertiary oxidation ponds ranged from (-) 65% to 21%, with a median removal of 0%. An evaluation of these results indicates that essentially no COD removal is accomplished in tertiary oxidation ponds. The form of chemically oxidizable organic matter may have changed from organic matter and bacterial cell debris in the influent to algae cells in the pond; but essentially no loss of oxidizable material, as measured by the COD test, occurred.

These results indicate that tertiary lagoons more closely approximate natural recycle than removal. This is the fundamental reason why lagoons are unable to achieve acceptable effluent quality and why it may be concluded that lagoons should not be considered equivalent secondary treatment processes.

3.5 OXIDATION POND AESTHETIC FAILURES

One hundred and ninety-seven cases of aesthetic oxidation pond failures were evaluated. These periodic

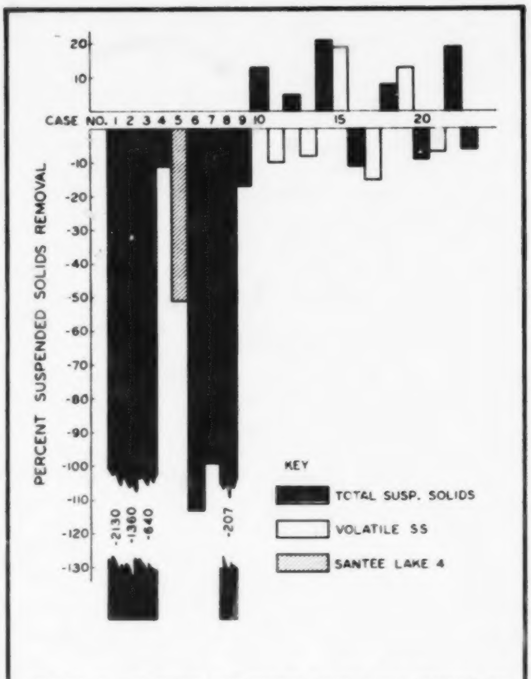

FIGURE 10 PERCENT SUSPENDED SOLIDS (SS) REDUCTION IN 23 CASES OF TERTIARY OXIDATION PONDS.

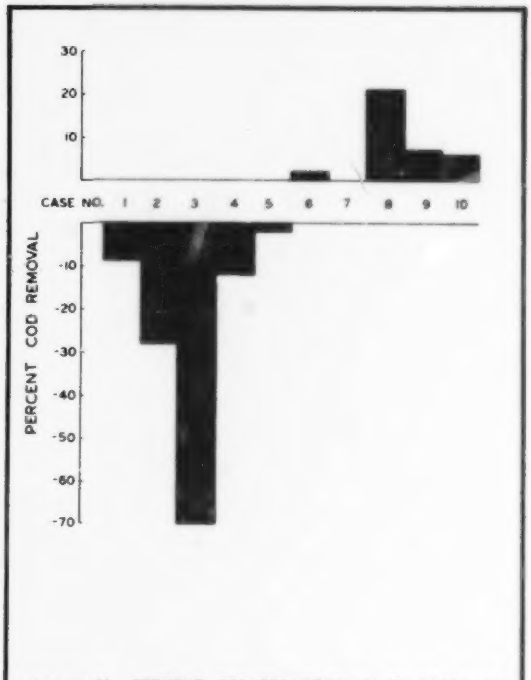

FIGURE 11 PERCENT COD REDUCTION IN 10 CASES OF TERTIARY OXIDATION PONDS.

TABLE 8

SUMMARY OF AESTHETIC FAILURES
197 CASES OF OXIDATION PONDS

TYPE	DESCRIPTION	NUMBER OF FAILURES	PERCENT FAILURES
A	Hydrogen Sulfide Odor	39	20
B	Algae "pig-pen" Odor	40	20
C	Septic Sewage Odor	55	28
D	Insect, Rodent Breeding	5	2
E	Marshy Weed Growth	7	4
F	Colored Effluent	18	9
G	Not Available	11	6
H	Not Reported	22	11
	TOTAL	197	100

aesthetic failures of lagoons by: 1) obnoxious hydrogen sulfide odors, 2) malodorous algae blooms, 3) noxious vegetative growth, 4) mosquito breeding, 5) highly colored effluents or 6) septic sewage odors; degrade esthetic qualities and appreciation. A summary of these aesthetic failures appears in *Table 6*.

In *Table 6*, it may be noted that 83% of the cases reported experienced periodic aesthetic failures. For 6% of the cases, no information was available regarding the occurrence or non-occurrence of aesthetic failures. The remaining 11% of the cases were reported not to experience aesthetic failures.

IV CONCLUSIONS

Based upon the findings of this research, the following conclusions may be drawn:

1. A method for evaluating both raw sewage and tertiary oxidation pond effluent quality was developed, based upon accepted effluent quality standards and expressed as "Degrees of Failure."
2. Regarding the effluent quality of raw sewage oxidation ponds:
    a. The effluent 5-day BOD quality greatly exceeded the accepted effluent BOD standards.
    b. The effluent suspended solids greatly exceeded the accepted effluent suspended solids standards.
    c. Oxidation ponds were found to be an in-effective and erratic treatment process for nitrogen, phosphorus and coliform reduction.
3. Regarding the effluent quality of tertiary oxidation ponds:
    a. The effluent 5-day BOD quality was not substantially reduced by tertiary lagooning and exceeded the accepted effluent BOD standards.
    b. The effluent suspended solids quality greatly exceeded the accepted tertiary standards and in most cases the effluent suspended solids concentration exceeded the influent concentration.
    c. The phosphorus removal over extended time periods was nil and in most cases the effluent phosphorus concentration exceeded the influent phosphorus concentration.
    d. The nitrogen removal was erratic. Organic nitrogen concentrations often increased during lagooning due to primary production (algae synthesis).
    e. The coliform reduction in tertiary lagoons was erratic.
4. The following oxidation pond aesthetic failures were found to occur periodically:
    a. Obnoxious hydrogen sulfide odors.
    b. Malodorous algae blooms.
    c. Noxious vegetative growth.
    d. Mosquito and insect breeding.
    e. Highly colored algae-laden effluents.
    f. Septic sewage odors.
5. Based upon the evaluation of effluent quality:
    a. In most cases studied, oxidation ponds did not significantly enhance water quality based upon reduction of suspended solids, volatile solids, chemical oxygen demand, nitrogen and phosphorus.
    b. Oxidation ponds which tend to recycle pollutants should not be considered equivalent to secondary treatment processes which remove pollutants.
    c. The continued use of oxidation ponds as presently designed and operated will not enhance, restore or maintain the quality of receiving waters.
6. The following recommendations are made for controlling oxidation pond effluent and aesthetic failures:
    a. Design and operation of "zero effluent" lagoons.
    b. Provide supplemental illumination and mixing followed by removal and wasting of suspended solids from the lagoon effluent prior to discharge.
    c. Substitute a treatment process for lagoons which meets the intent and objectives of the 1965 Water Quality Act.

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FIELD STUDIES ON STABILIZATION PONDS IN SOUTH INDIA

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Sewage treatment by stabilization ponds is best suited for moderate-sized communities, specially where cheap land is available. It is quite evident that the treatment method is of particular interest and significance to India. India has numerous towns and cities of moderate populations which have not so far enjoyed the benefits of sewerage and sewage disposal systems. Generally, the small towns in India can hardly meet the initial cost of the conventional sewage treatment processes. The sewage treatment by the stabilization pond method would make possible very substantial savings in capital cost and in maintenance, and its importance, therefore cannot be exaggerated. To obtain more information about treatment by stabilization ponds under the conditions prevailing in South India, the studies reported herein were undertaken. The major objective of the research was to investigate, under local conditions, the effect of depth and loading rates on organic matter removals, general pond performance, and coliform removals.

Several studies (1–6) have been reported in the literature regarding the effect of depth and organic loading on the efficiency of stabilization ponds.

DESCRIPTION OF EXPERIMENTAL PONDS

Four experimental stabilization ponds were constructed at the College of Engineering, Guindy, Madras, India, near the two existing anaerobic and stabilization ponds. All of the four ponds were made rectangular, 30 x 20 ft at the top. They were excavated into the ground with side slopes of 1½ : 1, and were enclosed by an earthen embankment with a slope of 1½ horizontal to 1 vertical. The purpose of the embankment was to keep off the rain water. A general plan of the existing and experimental ponds is shown in Fig. 1. Details of the experimental ponds are shown in Fig. 2. The water depth, surface area, and capacity of the ponds are given in Table 1.

The sewage discharges into the existing anaerobic pond (from which the effluent was led to the experimental stabilization ponds) was contributed by 700 residential and about 300 non-residential students and staff of the College of Engineering. Wide fluctuations in flow were common. Thus, in the summer months of May and June when the students were away, the sewage flow amounted to a small proportion of the flow occurring when the school was in session. The average sewage flow during the period of study was 24,000 gpd, with a maximum of 44,400 gpd and a minimum of 18,200 gpd. The high flow during the month of October was due to rains occurring at that time.

The sewage flowed into a sump from which it was pumped intermittently to the existing anaerobic pond for a total of 6½ hours daily on an average. From the anaerobic pond the sewage effluent was discharged into a stabilization pond (designated hereafter as "Pond A"), and the final effluent was disposed of into the fields. The existing anaerobic and stabilization ponds were 0.069 and 0.46 acre in area, respectively. Raw sewage was conveyed to the anaerobic pond in an open earthen channel terminating in two 3-in dia. pipes discharging freely. The anaerobic pond was separated from stabilization pond by a segmental earthen dike in which three 3-in dia. outlets were placed at equal intervals. The effluent from the stabilization pond was let out by two 2-in dia. hose pipes kept in the embankment opposite to the inlet pipes (Fig. 1).

The effluent from the anaerobic pond was made to discharge into the experimental stabilization ponds continuously by constructing a flow--split chamber 2.5 x 2.5 x 2.5 ft. The inlet to the chamber from the anaerobic pond consisted of a 2-in dia. galvanized iron pipe. The upstream end of the pipe was submerged about 6 in below the water level in the anaerobic pond to keep the floating matter, leaves, scum, and debris from entering the pipe, and the outlet was fixed into the chamber at a height 12 in from the bottom.

From the flow--split chamber four 3/4--dia. galvanized iron pipes were installed at the same level, 9 in above the chamber bottom. The ends of the pipes were connected to short lengths of 2-in dia. rubber hoses and the rate of flow through the pipes was controlled by 3/4-in dia. gate valves provided on each of the pipes.

The outlet from each experimental pond consisted of a 2-in dia. hose pipe placed centrally and at the opposite end of the inlet. The hose pipes were taken under the enclosing dike, and projected about 1.0 ft on the other side. The four outlets discharged the effluent into an earthen channel which carried it away to a nearby field for disposal.

METHOD OF OPERATION

The raw sewage was pumped to a measuring chamber provided with a 90° V-notch for measuring the rate of flow. A record of the daily flow of sewage and of the pumping schedule was maintained during the period of study.

The effluent from the anaerobic pond was fed to the experimental ponds through the flow-split chamber.
The flow was regulated by means of the gate valves, and was measured with the help of a container and a stop-watch. The experiments covered three periods of operation. The first, when the applied organic loading on all the four ponds was 230 lb BOD per acre per day; the second, when the loading was reduced to 170 lb BOD per acre per day; and the third; when the detention period was maintained at 9 days varying the applied loading rate.

SAMPLING PROCEDURES AND METHOD OF ANALYSIS

Samples of the influent and effluent of the experimental ponds were obtained periodically. The analysis was carried out more frequently for BOD, pH, and dissolved oxygen. The total suspended solids were determined occasionally.

The influent samples were first collected both at the outlet ends of the 3/4-in dia. pipes just before entering the experimental ponds, and also at the anaerobic pond just in front of the 2-in dia. inlet to the flow-split chamber. This was done to detect any appreciable difference in the BOD values between the two collection points. Since it was found from the analysis that the differences were of the order of 5 mg/1, the samples were subsequently collected only at the outlet ends. Samples of the effluent from the experimental ponds were collected in two bottles, one for dissolved oxygen determination, and the other for BOD, pH, total and suspended solids determinations.

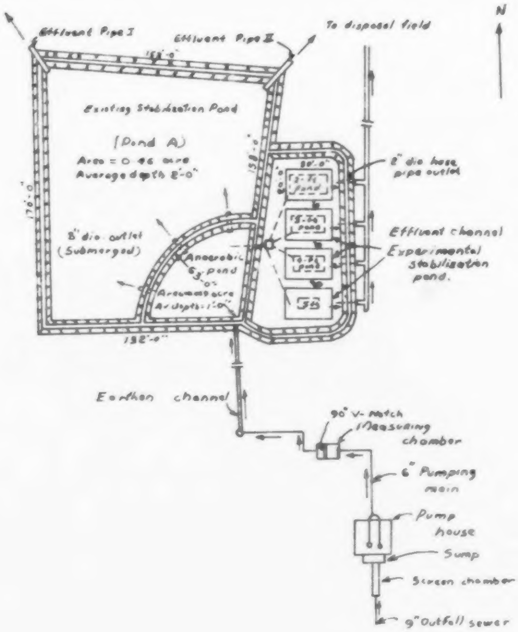
Chemical analyses were carried out according to "Standard Methods" (7). For dissolved oxygen determiation, the reagents were added in the field immediately after the collection of samples, and the azide modification of the Winkler's method was used to eliminate interference of nitrite nitrogen.

The BOD of the influent to the experimental ponds was determined as per "Standard Methods" on the unfilitered samples. In the case of effluents from the stabilization ponds, BOD values were obtained for both unfilitered and filtered samples. Filtration of the sample was done through Whatman No. 1 filter paper. The effluent from the ponds had a green color due to the presence of minute free-swimming algae which exerted oxygen demand when incubated in the BODtest. As a result, the effluent fluctuated widely in BOD and suspended solids because of the variation in numbers of algae present, rather than the amount of the original organic matter remaining. When the green algal material is removed from the pond effluents, it is reasonable to assume that the BOD of the sample is only associated with the remaining dissolved organic matter. Hence in evaluating the performance of all the stabilization ponds, only the BOD of the filtered effluent was considered.

For bacteriological analysis, influent and effluent samples were collected in 8-oz sterilized bottles separately following the standard collection technique. Both the total bacteria count and most probable number (MPN) of coliforms were determined at 37°C for 48 hours. The

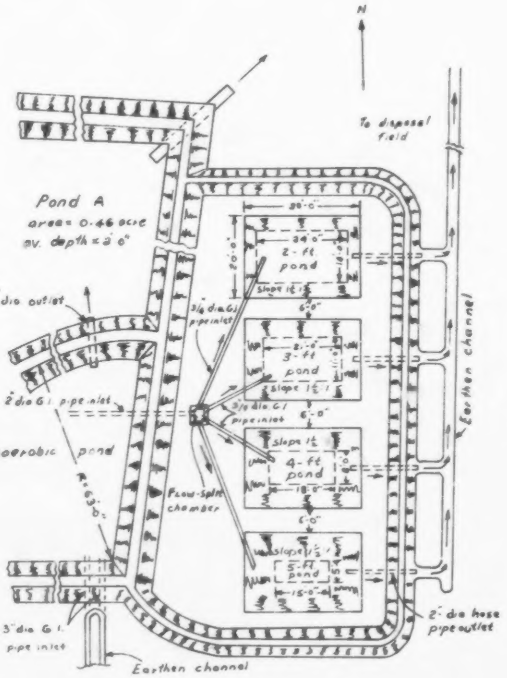
TABLE 1. Stabilization Ponds Details

Unit	Water Depth, ft	Surface Area, sq.ft.	Capacity, cu.ft.
Existing			
Anaerobic pond	2.75	3,000	8,250
Stabilization pond (Pond A)	2.00	20,000	40,000
Experimental			
2-ft pond	2.00	600	900
3-ft pond	3.00	600	1,170
4-ft pond	4.00	600	1,480
5-ft pond	5.00	600	1,750



Effluent Pipe I
Effluent Pipe II
To disposal field
N
152'-0"
Existing Stabilization Pond
(Pond A)
Area = 0.46 acre
Average depth 2'-0"
170'-0"
158'-0"
8" dia outlet (Submerged)
Anaerobic pond
Average acre
Av depth 1'-0"
132'-0"
2" dia hose pipe outlet
Effluent channel
Experimental stabilization pond.
Earthen channel
90° V-notch
Measuring chamber
6" Pumping main
Pump house
Sump
Screen chamber
9" Outfall sewer

FIG. 1 SITE PLAN OF EXISTING AND EXPERIMENTAL STABILIZATION PONDS



N
To disposal field
Pond A
area= 0.46 acre
av. depth = 2' 0"
30'-0"
24'-0"
20'-0"
2-ft pond
Slope 1½:1
6'-0"
3/4" dia outlet
3/4 dia G.I pipe inlet
3-ft pond
Slope 1½:1
2" dia G.I. pipe inlet
3/4 dia G.I pipe inlet
Slope 1½
4-ft pond
18'-0"
Anaerobic pond
Flow-Split chamber
Slope 1½:1
5-ft pond
15'-0"
Earthen channel
3" dia G.I. pipe inlet
Earthen channel
2" dia hose pipe outlet

FIG. 2 EXPERIMENTAL STABILIZATION PONDS

TABLE 2.

Influent 5 — Day 20° C BOD During Various Periods

Sample No.	First Period (At 230 lb per acre per day)	Second Period (At 170 lb per acre per day)	Third Period (At 9 Days Detention Period)
		BOD, mg/1	
1	200	---	90
2	140	180	90
3	140	140	100
4	200	140	110
5	130	160	100
6	150	180	110
Average	160	160	100

intensity of phyto- and zoo-planktons was enumerated on four different occasions.

PERFORMANCE OF EXPERIMENTAL PONDS

The BOD of the influent (anaerobic pond effluent) to the experimental ponds varied from 90 to 200 mg/l during the study period with an average of 160 mg/l and 100 mg/l for the first two periods of operation and the last period, respectively (Table 2).

The results of the effect of organic loading of 230 lb BOD per acre per day on the experimental stabilization ponds are given in Table 3. The BOD removal varied from 64.0 to 85 percent during the period. Highest average removal of 78.2 percent was obtained in the 2-ft pond decreasing to 72.2 percent in the 5-ft pond. The 3-ft and 4-ft ponds effected average BOD removals of 76.2 and 78.0 percent respectively. Student's t-tests of significance of the difference were not significant. Hence, it is difficult to say whether pond depth had any influence over the efficiency of treatment at the loading of 230 lb BOD per acre per day.

The relationship between the influent BOD and effluent BOD at this organic loading was also investigated. With increase in values of influent BOD there was an increase in the BOD of the effluent and so the correlation was positive. Theoretically, there should be some correlation between the influent and effluent BOD values, only if the samples collected correspond to the influent. That is, if the detention period in the pond is (say) 2 days, the effluent on a particular day should have reflected the influent entered the pond 2 days before. But in this case, the influent and effluent samples were collected on the same day. Hence, the correlation obtained by the analysis of the influent and effluent samples as collected in the present studies does not reflect the correct conditions, but is offered merely as suggestive of the relation between influent and effluent BOD values.

When the organic loading was reduced to 170 lb per acre per day (second period of operation), the BOD removals ranged from 67.5 to 87.5 percent. Maximum average removal was 80.9 percent in the 4-ft and 5-ft ponds and the minimum was 78.5 percent in the 3-ft pond (Table 4). These results showed a definite improvement in the BOD removal efficiencies as compared to those obtained at loading of 230 lb BOD per acre per day. Student's t-test indicated that the differences between the mean percent BOD removals of the samples were not significant. Therefore, it is not possible to conclude that the depth had any effect on performance at the loading of 170 lb per acre per day.

From the analysis of the relationship between the influent and effluent BOD values, the coefficients of correlation were found to be negative. This proves that a correlation between the influent and effluent BOD values is not correct without considering detention time.

The significance tests of the differences in mean percent BOD removals in the various ponds at 230 and 170

TABLE 3.

Performance of Stabilization Ponds at Organic Loading of 170 Lb. BOB Per Acre Per Day

Unit	2-ft pond	3-ft pond	4-ft pond	5-ft pond
Detention Time, Days	2.8	3.7	4.6	5.5
Sample No.		* BOD Removal, percent		
1	74.0	---	66.5	72.0
2	82.5	85.0	---	---
3	---	72.5	72.5	64.0
4	78.0	76.0	76.0	72.0
5	75.0	72.5	75.0	75.0
6	81.3	75.0	75.0	78.0
Average	78.2	76.2	73.0	72.2

* Based on filtered effluent BOD values.

TABLE 4
PERFORMANCE OF STABILIZATION PONDS AT ORGANIC LOADING OF 170 lb BOD PER ACRE PER DAY

Unit	2-ft pond	3-ft pond	4-ft pond	5-ft pond
Detention Time, Days	3.7	4.2	6.2	7.3
Sample No.	* BOD Removal, percent			
1	---	70.0	67.5	---
2	82.0	---	80.0	80.5
3	[illegible]	70.0	---	67.5
4	81.0	80.0	82.0	81.3
5	75.0	87.5	87.5	87.5
6	82.0	85.0	87.5	87.5
Average	79.8	78.5	80.9	80.9

* Based on filtered effluent BOD values

lb per acre per day showed that the differences were not significant, and hence the loading had no effect on the organic removal efficiencies in the ponds.

Fig. 3 shows the relationship between depth and BOD removals at loadings of 230 and 170 lb per acre per day. The average percent removal at 230 lb BOD per acre per day loading decreased from 78.2 to 72.2 as the pond depth was increased. When the applied load was 170 lb per acre per day, the average BOD removal decreased from 79.8 to 78.5 percent between 24 and 36 inches depth beyond which it recorded an increase.

During the third period of operation, the flow was adjusted to give a theoretical detention period of 9 days. The loadings and the percent organic removals are given in Table 5. The BOD removals varied from 64.0 to 88.0 percent. The average removal of BOD was more or less near about 79 percent at different loadings. The results of significance tests showed that the difference between the mean removals of the samples were not significant.

Fig. 4 shows the effect of organic loadings on the performance of the experimental ponds. Similarly, the relationship between detention time and BOD removal is indicated in Fig. 5. The percent BOD removal decreased with increased loading in the case of the 3-ft pond. This was not the case with the 2-ft, 4-ft, and 5-ft ponds which gave higher organic removals at 170 lb per acre per day loading. Detention time from 2 to 9 days did not show any influence on the BOD removals (Fig. 5).

Variation in Dissolved Oxygen

Dissolved oxygen was always absent in the anaerobic pond effluent (influent to the experimental ponds), though some algal organisms were present in the pond on a number of occasions.

In the case of the stabilization ponds, dissolved oxygen was found to be present on most occasions. When the loading was 230 lbs per acre per day the dissolved oxygen ranged from 0 to 31.8 mg/l The highest value of 31.8 mg/l, 400 percent saturation, was observed in the effluent of the 3-ft pond at 3 P.M. on a particular day and the corresponding values were 25.6 and 24.0 mg/l in the 4-ft and 5-ft ponds, respectively.

One set of 24-hour test was done to study the diurnal variations in dissolved oxygen. The results are furnished in Table 6. Dissolved oxygen found to increase in the effluent from the ponds with increase in intensity of sunlight, and to reach a maximum at about 2 P.M. The maximum value reached in pond A was 18.0 mg/l at 12 Noon. In the experimental ponds, the maximum value was 23.0 mg/l in the 5-ft pond. Dissolved oxygen in the effluents was found to decrease after 2 P.M., and it completely disappeared during the night. The peculiarity observed in the case of the 2-ft pond was that dissolved oxygen was absent as early as 2 P.M. on that day.

Complete disappearance of dissolved oxygen at night at the loadings adopted may be due to the utilization of

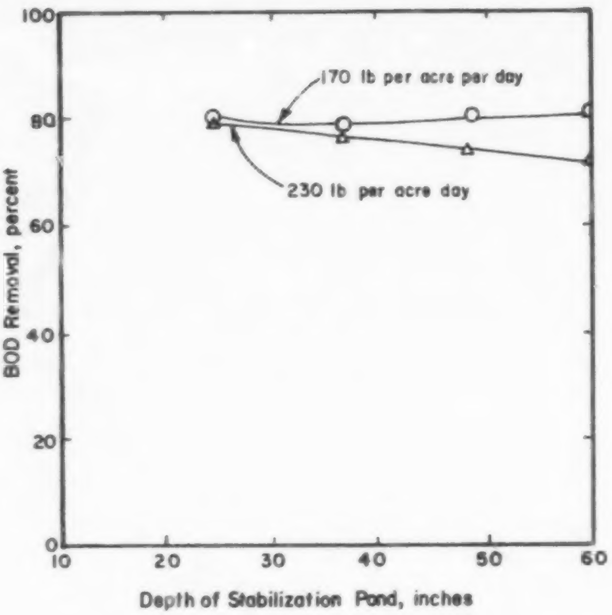


FIG. 3 RELATIONSHIP BETWEEN DEPTH AND PERCENT BOD REMOVAL IN STABILIZATION PONDS

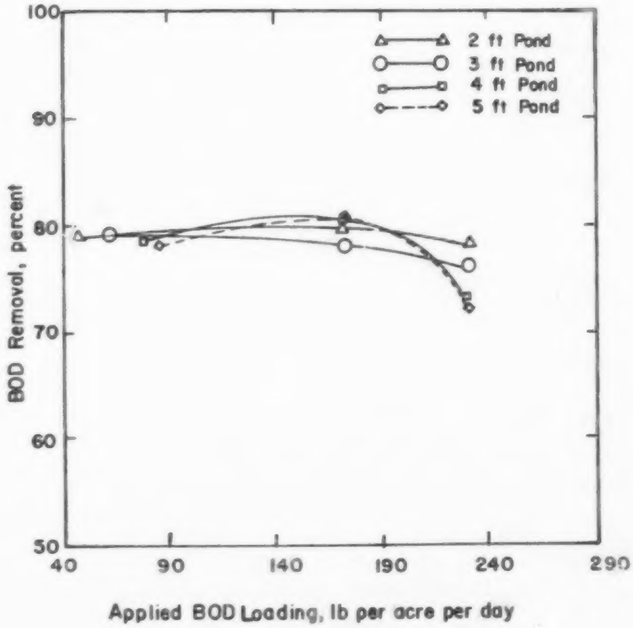


FIG. 4 EFFECT OF LOADING ON THE EXPERIMENTAL PONDS

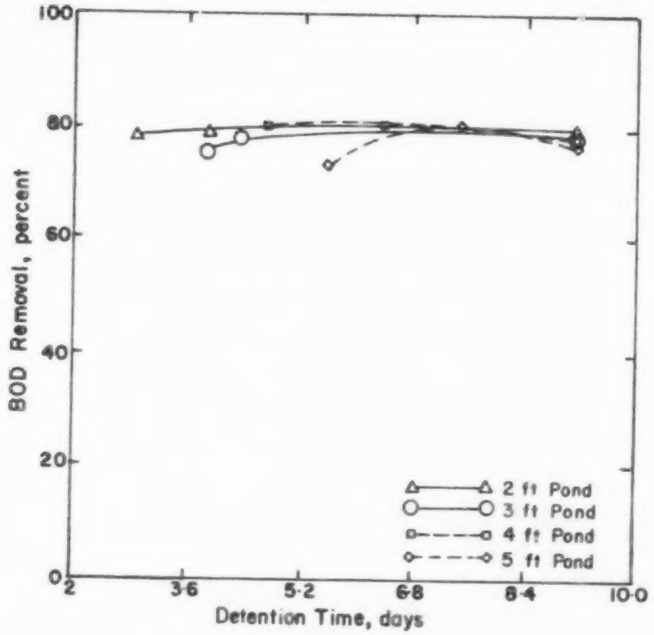


FIG. 5 RELATIONSHIP BETWEEN DETENTION TIME AND BOD REMOVAL

Time, hours	Dissolved Oxygen, mg/l				
	2-ft Pond	3-ft Pond	4-ft Pond	5-ft Pond	Pond A
0600	0.0	0.0	0.0	0.0	1.0
0800	0.0	1.2	2.4	2.3	2.9
1000	1.5	3.0	4.5	15.4	12.0
1200	2.2	5.2	6.4	22.6	18.0
1400	0.0	12.4	21.2	23.0	12.2
1600	0.0	6.0	18.6	18.8	6.0
1800	0.0	2.6	7.4	5.8	3.0
2000	0.0	0.4	1.0	0.0	2.0
2200	0.0	0.0	0.0	0.0	1.5
2400	0.0	0.0	0.0	0.0	1.0
0200	0.0	0.0	0.0	0.0	0.0
0400	0.0	0.0	0.0	0.0	0.0
0600	0.0	0.0	0.0	0.0	0.0

TABLE 6.

Diurnal Variation of Dissolved Oxygen

TABLE 5.

Performance of Stabilization Ponds at a Detention Period of 9 Days

Unit	2-ft pond	3-ft pond	4-ft pond	5-ft pond
Loading, lb BOD per acre per day	45	60	75	85
Sample No.	* BOD Removal, percent			
1	88.0	88.0	82.0	82.2
2	84.0	80.0	88.0	79.8
3	74.5	81.4	84.0	80.0
4	76.0	76.0	64.0	80.0
5	77.7	76.0	80.0	80.0
6	74.4	72.6	74.0	68.0
Average	79.1	79.0	78.7	78.3

* Based on filtered effluent BOD values.

oxygen by algae for respiration, and also to the biochemical oxygen demand exerted by the microorganisms. The afternoon samplings showed supersaturation of dissolved oxygen due to photosynthetic oxygen production.

Coliform and Bacterial Reduction

The average reduction of the total bacteria count between the influent and effluent from the experimental ponds was in the order of 93 to 96 percent even at the short detention period adopted during the study. The MPN of coliform organisms in the influent ranged from 70 x 10^6 to 220 x 10^6 per 100 ml. In the effluents the values ranged from 0.08 x 10^6 to 28.0 x 10^6 per 100 ml. The low values occured after rains. Parker *et al* (1) found that the coliform count in the anaerobic pond effluent was 25 x 10^6 per 100 ml. The MPN of coliform organisms reported by Neel and Hopkins (8) was 1.075 x 10^6 per 100 ml. The coliform removal in the experimental ponds varied from 93 to 99 percent. Similar coliform reduction in stabilization ponds have been reported by many others (1,5,6,0). The student's t-test showed that the differences in mean percent removals between the various ponds were not significant. It may be concluded that depth has no effect on the removal of coliform organisms when measured in terms of reduction in the MPN.

Occurrence of Phyto- and Zoop -Planktons

The predominant planktons observed in the experimental ponds during the study period are shown in Table 7. The important algal species were found to be *Eudorina, Oocystis, Pandorina, Merismopedia, Oscillatoria,* and *Spirulina,* while the zooplanktons included such as *Cyclidium, Paramecium,* and *Brachionus.*

TABLE 7.

Planktons in Experimental Ponds

Organic Loading lb BOD per acre per day	Algal Species	Zooplanktons
[illegible]	Eudorina	Vorticella
	*Pandorina	*Paramecium
	*Oscillatoria	[illegible]
	[illegible]	[illegible]
	Euglena	*Cyclidium
	Ankistrodesmus	[illegible]
	*Merismopedia	
	*Spirulina	
	Cyclotella	
[illegible]	*Eudorina	[illegible]
	*Pandorina	*Cyclidium
	*Oocystis	*Paramecium
	Scenedesmus	*Brachionus
	*Merismopedia	
	*Oscillatoria	
	*Spirulina	
	Euglena	
	[illegible]	
	Cyclotella	
	Navicula	

* Predominant species

It appears likely that as the organic loadings are increased, zooplanktons become more prevalent.

CONCLUSIONS

From the experiments conducted the following conclusions may be drawn:

1. The stabilization ponds can be operated at as high as four times the conventional organic loading of 40-50 lb BOD per acre per day where the climatic conditions are favorable.
2. Pond depth was found to have no influence on the operating efficiency under the conditions studied.
3. Detention period from 2 to 9 days did not show any effect on the BOD removal efficiency.
4. Dissolved oxygen was present in the pond effluents during the daytime, but it was completely used up due to the algal respiration and bacterial decomposition during night. The dissolved oxygen concentration reached as high as 400 percent supersaturation.
5. The reduction of bacteria and coliforms was not affected by the depth and the detention time.
6. There was no correlation found between the organic loading rate and the predominant phyto- and zoo-planktons.

ACKNOWLEDGEMENTS

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A DECADE OF STABILIZATION LAGOONS IN MICHIGAN WITH IRRIGATION AS ULTIMATE DISPOSAL OF EFFLUENT

by

*Earl A. Myers & T. C. Williams***

Over the past ten years, Williams & Works has designed 28 stabilization lagoon type sewage treatment systems for Michigan communities. During the coming year we are scheduled to do 15 more. All of the systems have been designed with nominal loadings of 20 pounds BOD/day/acre of water surface and most of them have been designed for semi-annual discharging in accordance with Michigan Department of Public Health standards. Several of the completed lagoon systems and all those presently being designed involve irrigation of the treated effluent.

The advent of official concern, and subsequent regulations regarding the degree of phosphorus that must be removed before waste water is discharged into streams or lakes, was the primary motivation behind the irrigation of effluent in Michigan. Irrigation, as an engineering tool used for abating pollution, permits design flexibility for proximity of treatment plant to population area, for maintaining quality of water bodies in the region, for incorporating topographical features into treatment facility design and for potential reuse of the renovated effluent. Before discussing the use of irrigation for ultimate disposal of lagoon effluent, it seems appropriate to relate some of our successes and failures with stabilization ponds to date.

Most of our lagoon systems have been multi-celled, similar to the design used at Belding. There a 50-acre five-cell arrangement designed for a population of 5,000 is in use. The first cell is anaerobic. The second cell is a 20-acre facultative pond followed by a series of one 15-acre and two 7.5-acre ponds. The ponds at Belding have been functioning quite well for about five years on a twice-a-year drawdown basis. Their operation will be modified this fall to include the irrigation of effluent as part of a FWQA research project.

The project will span three years and cost approximately $200,000. There are four primary objectives:

> To establish more specific criteria for spray irrigation of chlorinated sewage stabilization lagoon effluent utilizing the "Living Filter Concept" advocated by Penn State.
>
> To develop design parameters for irrigation of stabilization lagoon effluent on glacial soils.
>
> To demonstrate conservation of resources by using the nutrients and waste water for growing trees and sod to meet municipal needs.
>
> To study effects of rapid recirculation of pond effluent for odor control during the spring break-up.

Compared to other treatment methods, stabilization lagoons are less expensive to install and are more economical to operate. All of our ponds to date have been designed primarily for handling municipal sewage and are natural oxidation ponds except Carson City and Saranac where floating aerators are used. Most of our early problems with stabilization lagoons are now adequately covered by design criteria, however, a few still deserve to be mentioned.

Many of our designs incorporated an anaerobic first cell into the system. We have had several sad experiences trying to get the reaction started in some of these instances when they were underloaded and no seed sludge was used. Once they began working, however, there was no odor problem except when a scum layer was allowed to form. The break-up or removal of the scum layer caused the odor to disappear. The behavior of subsequent ponds indicates that 30-40% BOD removal is attained in the anaerobic cells. On this basis we feel that anaerobic cells frequently have a place in stabilization lagoon systems.

Sealing of lagoon bottoms and sides has been an interesting, and sometimes aggravating, problem. At several locations the cost of securing, hauling and placing clay seals has encouraged consideration of alternatives. For one installation we tried asphalt prime and seal. It was a total failure. We recommend that you do not try it unless there is sufficient water readily available to fill the pond immediately after sealing. Leakage problems may also be encountered when bentonite is used. This is especially true when use of lagoons is begun with insufficient early hook-ups and minimum flows into the system. Erosion can cause the loss of side seals and weeds can cause leaks in bottom seals. A series of four-inch diameter muskrat holes through the side seals can encourage excess seepage to say the least.

We have had some other interesting failures. (It is preferred to make certain that failures are "interesting" otherwise they become "expensive mistakes.") One was an attempt to draw off specific layers of final pond water and another was an attempt to attain reproducible results of effluent quality. Quality data obtained from Michigan oxidation lagoons have been summarized and presented earlier this week by Mr. M. S. Richmond (6) of the Michigan Department of Public Health. Thus, no comments on quality will be included in this paper, which will concentrate primarily on irrigation of effluent.

The ultimate disposal of effluent from lagoons used for stabilizing municipal sewage has been of prime concern in Michigan since 1961. That was the first year this type of treatment facility was sanctioned by the Department of Public Health. To date, the standard procedure has been for no planned discharge from the lagoons except during very brief periods in late fall and early spring. These periods were chosen to coincide with maximum stream flows and minimum stream use. Since the quality of the discharge from properly designed, constructed and managed lagoons is generally equivalent to discharge from secondary treatment plants, this twice-a-year drawdown procedure has been quite acceptable. The fact that installation and operational costs are minimal also, made this method of disposal attractive.

A recent Michigan Water Resources Commission stipulation states that by December, 1972, communities with populations greater than 1,500 must remove 80% of the phosphorus entering the sewage treatment facility before the effluent may be discharged into a stream. This stipulation poses a problem and a challenge to all present and prospective users of stabilization lagoons. The cost and difficulty of chemical precipitation, of microscreening or of any other known method for the removal of phosphorus from algae laden lagoon discharge, frequently is prohibitive. This has caused many to consider irrigation as a means of ultimate disposal. For this reason it is considered appropriate to discuss some of the many ramifications associated with the irrigation of stabilization lagoon effluent.

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First of all, it is important to document the concept. Proceedings of the 1968 Symposium on Municipal Sewage Effluent for Irrigation held at Ruston, Louisiana have been published by Wilson and Beckett (8). Their 169-page booklet presents many of the nationwide aspects of the subject. Also, extensive research has been conducted at The Pennsylvania State University using spray irrigation of secondary treated and chlorinated municipal sewage effluent. A variety of crops grown on several soil types were irrigated at specified application amounts and detailed results of the degree of renovation recorded. These data are reported in Penn State Studies No. 23 by Parizek, et al (5).

This paper will relate some of the specifics of the Penn State study to the practical application of irrigating effluent from stabilization lagoons for municipalities in Michigan. The basic concept of waste water renovation and conservation advanced by Penn State and being advocated for Michigan can briefly be outlined as a 4-R cycle of return, renovation, recharge, and reuse.

The effluent after receiving secondary level of treatment and chlorination is returned uniformly over the land through an irrigation system. The top soil layers in conjunction with growing crops renovate this waste water. That is, the nutrients - phosphorus, nitrogen, etc. - are removed from the effluent by biological, chemical and mechanical action. The renovated water not used by the crops may move downward into the groundwater reservoir or may move laterally over a restricting layer and be outletted into a surface water course. In either situation, the water is cleaned to such an extent that it is usable for agricultural, industrial or domestic purposes.

Adequate balancing of the biological and hydrological systems alluded to in the 4-R cycle can be attained through proper engineering and careful management of the renovation system. More specifically this means choosing the proper irrigation application rate in inches per hour and application amounts of inches per week and per year for the crops, soils topography, and geology of the area being irrigated.

Data in the following table, presented by Kardos (3), includes typical mean concentrations and total amounts of various constituents found in secondary treated effluent. While these data were from the combined flows of a trickling filter and an activated sludge plant, they do not differ appreciably from the data for quality of effluent from stabilization lagoons reported at this meeting by M. S. Richmond.

The primary purpose of crops in the renovation cycle is to remove the nutrients added through the irrigation of effluent. Once the nutrients are incorporated into the crops, it is imperative to remove these crops from the irrigation area so the renovation zone does not become less efficient with use. Due to the recent Water Resource Commission stipulation, phosphorus is the nutrient receiving the most consideration at present. There are many research data available stating that this element can readily be removed by soils and crops (2, 3, 5, 8). Thus, if phosphorus is the primary consideration, the choice of crop can vary with the typical land use pattern of a community. That is, at Jackson where there is dairy farming, corn and grass can be grown; at Harrison where lumbering prevails, trees are appropiate; at East Jordan, irrigation of the airport's grass runways would seem logical; and at Denton, Timothy hay for the horses used for recreation could be a good choice. While crops frequently remove only 25-50% of the phosphorus applied, this poses no problem since most soils have a very high capacity for absorbing phosphorus.

The nitrates contained in the effluent are not as readily removed as the phosphorus. Therefore, the overall design must insure that these nitrates are adequately removed by crops and/or soil microbes so that the renovated water does not cause undue degradation of the groundwater. A 100-bushel per acre corn crop requires approximately 200 pounds of nitrogen. Comparison of this crop requirement with the typical amount supplies through 60 to 70 inches of irrigated effluent per season, as presented in the table, indicates effective nitrogen removal can be obtained. During one year of research, more than three times the amount of nitrogen supplied by 100 inches of effluent were removed through the harvesting of the Reed Canarygrass during that year. Kardos reported numerous other comparisons of amounts of nutrients added to crops by irrigation of effluent to amounts removed by harvested crops.

Coerver (1) categorized and tabulated the responses of 45 states to questionnaries concerning health regulations regarding the use of domestic sewage. The regulations vary extensively, but in most instances irrigation of effluent is permitted on crops other than those to be eaten raw by human beings.

Farmers ordinarily irrigate the best soils or fields on their farms. The same should also be true for municipalities irrgating effluent from stabiliazation ponds. Unfortunately, too many make the sad choice of acquiring cheap land and a modification of the old saying "penny wise but damn foolish" is borne out. When choosing an irrigation site, the best farm land available should be the first choice.

The higher the sand content, generally, the faster the infiltration. However, the greater the clay content the better the renovation, provided the water infiltrates and percolates through the soil. The best all around average seems to be soil of loam or sandy loam texture. The sandy loam soils are somewhat poorer in inherent nutrients and more droughty than preferred for regular agricultural purposes. Since the nutrients and water are supplied by the effluent, these soils are excellent for irrigating with municipal sewage effluent.

Recommendations for minimum soil depth requirements vary from two feet to hundreds of feet. Since the greatest percentage of the roots are in the top one or two feet of the soil, a two-foot depth is adequate if all other criteria are met. To compensate for less than ideal soil type and uniformity, to provide for additional phosphorus

Mean concentration (mg/l) and total amounts (lb/a) of various constituents in waste water applied to the Agronomy Area at a rate of two inches per week 1963-1966.*

Constituent	Concentration (mg/l)				Amounts Applied (lb/a)			
	1963	1964	1965	1966	1963	1964	1965	1966
MBAS[1]	3.23	1.82	0.74	0.28	35.2	27.2	9.7	4.1
Phosphorus	9.70	8.95	6.60	5.21	105.6	133.8	86.8	75.5
Nitrate-N	5.80	13.80	5.50	7.44	63.4	206.6	72.3	107.9
Organic-N[2]	6.90	4.40	2.60	5.08	75.4	65.2	34.2	73.7
Potassium	17.3	16.0	19.2	18.1	188.2	239.2	252.5	262.5
Calcium	31.5	30.1	28.8	27.1	342.8	450.2	378.7	393.0
Magnesium	18.6	15.8	15.3	17.5	202.4	236.4	201.2	253.8
Sodium	46.7	32.9	34.6	37.7	358.0	492.2	455.0	546.7
Chloride	43.50	42.30	43.20	49.22	473.2	632.8	568.1	713.7
Boron	--	0.4	0.32	0.32	--	6.0	4.2	4.6
pH	7.3	7.2	7.6	7.7	--	--	--	--
Waste water applied (in.)					48	66	58	64

[1] Methylene blue active substance.

[2] Includes NH_3-N.

*From Kardos (3).

absorbing capacity, and to assure a well aerated deep zone for maximum microbial activity for nitrate renovation, one can usually say the deeper the better for a particular soil type. A very sandy soil that is several hundred feet deep, however, may not be as good as a loam which is four to six feet deep.

Schwartz & Bendixen (7) state that a minimum critical depth of unsaturated zone exists somewhere between two and four feet and that less depth results in a decrease of treatment capacity. They state that an optimum depth would be a least five feet.

Proper choice of irrigation rates and amounts are essential in any system. Even when these are determined from the best possible information, it is judicious to plan for flexibility as the system is used and inflow volumes vary from year to year. Generally, application rates range from 1/8 to 1/3 inches per hour, whereas application amounts range from 1 to 3 inches per week and from 50 to 100 inches per year. Numerous possibilities exist in irrigation equipment which may be used. Types of systems vary extensively – one may have a portable pump and power unit, and portable aluminum laterals as at the 50-bed Ottawa Infirmary Lagoon Treatment System, – another may be a permanent system with fully equipped and permanently constructed pump house, with buried mains and laterals, with permanent risers and sprinklers, and with an automatic electrical valve controller for a design population of 3,000 as at Middleville – others could have any combination in between, as is planned for the townships surrounding Jackson with an initial design population of 36,000 and a future population estimate of

100,000.

Since irrigation is a large and diverse field in itself with more than 75 years of substantial engineering background, I will merely draw a simple analogy. "just as one does not hire a butcher when he needs a surgeon, one should not hire a plumber when he needs a professional irrigation engineer".

Topography frequently affects the type of irrigation system as well as the agricultural management practices one chooses to use. When it is convenient to place the stabilization lagoons at an elevation sufficiently above the irrigation area, it might be more economical to flood irrigate an area than to use sprinklers. We have two systems using the surface flooding method presently under construction and two more are being designed. Several advantages of flood irrigation are that gravity is used to distribute the water rather than pumps, and that no chlorine is required for flood irrigation at the present time in Michigan. Therefore, there is no need for chlorine equipment, a separate chlorine room in the pump house, or a 20-minute detention tank for chlorine contact time. Thus, the costs are decreased appreciably.

These advantages must be considered in light of some necessary precautions: to insure reasonable distribution, the land to be irrigated must be leveled or at least planed smooth; for flooding at a rate higher than the infiltration rate of the soil, large distribution pipes are needed; to adjust flow rates to meet soil conditions, adequate design flexibility and good management are required; and to maintain and operate some systems, a fair amount of labor is required.

To assure consistent removal of phosphorus and nitrogen, it is generally felt that the effluent must infiltrate and percolate through some soil depth. Once it has been adequately renovated it may continue to move downward to the groundwater table or move back out of the ground and off the area through surface channels. Irrigation rate (inches per hour); topography; and soil texture, structure and stratification play important roles in determining the ultimate disposition of the renovated effluent. One must not irrigate areas where water comes back out or the soil as interflow or hillside seepage. Interflow combined with excessive application rates may cause erosion on steep slopes or under certain soil conditions even on long slight slopes. Adequate standard agricultural conservation measures must be practiced if the renovation zone is to maintain its efficiency.

Knowledge of the geology under an irrigation area is of utmost importance in determining the course of the renovated effluent. Parizek and Myers (4) present appreciable information concerning this area in their paper "Recharge of Ground Water from Renovated Sewage Effluent by Spray Irrigation".

Borings on site are necessary and logs of wells in the vicinity of the irrigation area are helpful in determining groundwater depth, direction of flow, etc. When the site borings are taken, it is of great benefit to place drop pipes and screens for monitoring water depth and quality both before irrigation begins and at regular intervals throughout the entire year, even if irrigation is confined to certain parts of the year. Adequate renovation and quality control can be assured only by sufficient monitoring.

Management is the key to irrigation of effluent becoming a significant tool for the conservation of nutrients and renovated water, and for the elimination of much pollution in our present water bodies. Proper management of the crops and of the irrigation rates and amounts can provide the biological balance required. When astute management of soils is added to the above, adequate hydrological balance can also be attained.

The development of stabilization lagoon designs at Williams & Works is the result of team effort. This effort involves our planners, groundwater geologists, soil scientist; water quality specialists, and agricultural, electrical, mechanical, and sanitary engineers. With adequate engineering, conscientious management, and mature and reasonable cooperation among all persons and agencies involved, it can be concluded that irrigation as ultimate disposal of stabilization pond effluent can be carried out to meet all public health requirements and all reasonable surface and subsurface water quality standards.

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A DECADE OF RALSTON PURINA LAGOON EXPERIENCE

C. B. Smith
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1. *Introduction*

You are interested in economical waste treatment. Profit margins are so low in the food business economical waste treatment is vital. It's a pleasure to review our success with lagoon treatment. Success -- to you and I this means

A High BOD removal, 95% plus.
B Dissolved oxygen in effluent.
C High coliform removal.
D Approval of governmental authorities;
F Low first and operating cost.

2. *Ownership of Lagoons*

We own most outright. In a few cases municipalities or a private sewage company have built facilities adjacent to our plants. In those now owned we pay for amortization and operating costs. Accordingly, we are very much concerned that an economical method of treatment is selected.

3. *Types of lagoon systems*

Ralston Purina has many types of systems. One type is where we have primary treatment, that is, a settling and flotation tank followed by an aerobic lagoon.

Another consists of primary treatment then an anaerobic lagoon and after that aerobic lagoon.

Still another type of system consists of an anaerobic digestion cell followed by an aerobic lagoon. I might stop to explain here that we would consider an anaerobic lagoon one of sufficient depth to exclude the effects of sunlight. It would also be loaded rather heavily relative to its volume so that only anaerobic bacteria would exist in the lower levels. Typical loading would be 3 lb. B.O.D./1000 ft. 3/day. This would give a surface loading of 1000 lb./acre/day. Retention period would range from 5 to 10 days.

The aerobic lagoon, of course, would have a depth not in excess of 6 feet. The BOD loading would normally not be in excess of 100 pounds per acre and many less than 50 pounds BOD per acre.

We also have primary settling basins followed by power aerated aerobic lagoons then natural aerobic lagooning then a chlorination basin, then power aerated tertiary aerobic lagoon as final treatment.

We have still another type where the lagoon is simply power aerated aerobic.

4. Ralston Purina Company either owns or has adjacent to its properties 21 lagoon systems.

5. Several types of wastes are treated. They are:

Chicken dressing plant wastes
Turkey dressing plant wastes
Rendering plant wastes
Dairy wastes
Mink Chow mixing plant wastes
Hog wastes
Dog wastes
Cat wastes
Human sewage from processing plants

6. Why do we use lagoon treatment whenever we are able to do so? First of all, they are low in first cost. They are low in first cost providing that suitable land is on hand or can be purchased at a reasonable price. For lagoons more area is required than with other types of treatments. Then, too, having the land is not the total answer. The geological structure must be proper. The lagoon must hold water. It can't be located over sand and gravel strata or rock strata that would leak. Another thing that must be considered is the topography. It is usually expensive to install a lagoon system in territory that is too hilly or that contains rock at a shallow depth.

Another advantage of lagoons is the degree of treatment that can be achieved. Tests as high as 98% BOD reduction with effluents running in the neighborhood of 13 parts per million dissolved oxygen have been made.

Another advantage of the aerobic lagoon is its lack of odor. Properly loaded they smell no better or worse than any normal lake.

Another marked advantage is very low maintenance cost. About all that is required is testing and mowing of the banks. Of course, with any other type of treatment you have the tests also.

Another advantage to our company is that these lagoons can be a source of cooling water for condenser operation.

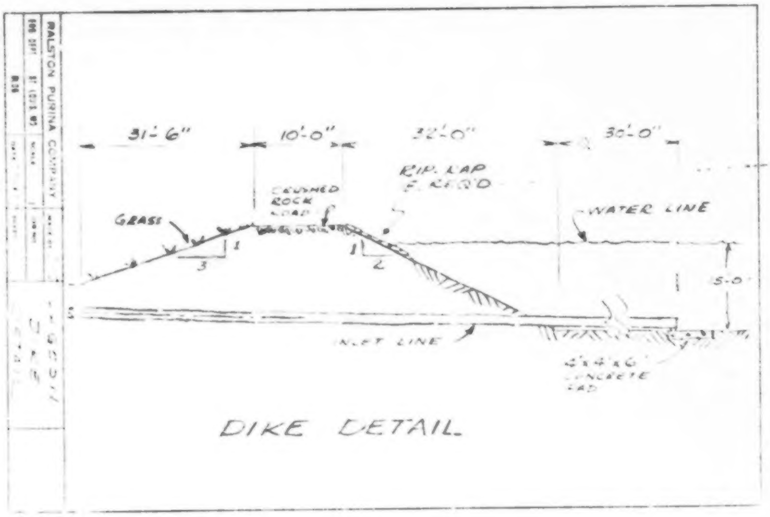
Still another advantage is the ability of the lagoon system to take a slug of overload in case of a breakdown of the primary treatment device or if the plant personnel for some reason does not do a proper dry cleaning or screening job. Incidentally, we have had some of our lagoons operating with good efficiency far beyond normal rating for extended periods. For instance, the lagoons at one plant have run at loadings of 150 pounds BOD/acre and still had a green color with good records on dissolved oxygen without any problems whatsoever on odors.

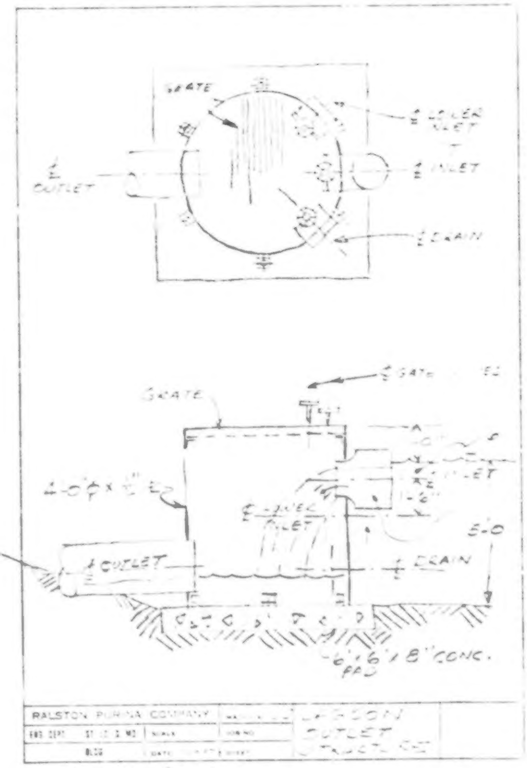
7. Selected Details:

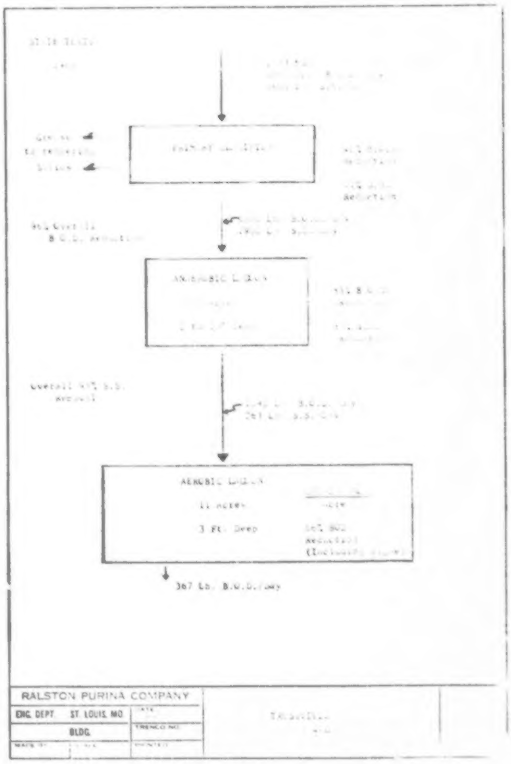
Now, gentlemen, if we can switch to the slides there are several details that you will be interested in.

First, let's discuss the dikes. For the most part we have followed standards as developed by our progressive and helpful Missouri Water Pollution Board. As you note, the water side of the dike is built on a slope of 1 to 2. The berm is a minimum of 10 feet wide so that a vehicle can be used to inspect the dike. A 1 to 3 slope is used on the other side of the dike so that the grass can be readily mowed.

We have not found it necessary to extend the inlet pipe to the center of the lagoon. A few feet away from the edge of the bottom dike line is sufficient. Frankly, I would recommend not going any further than this


RALSTON PURINA COMPANY
ENG DEPT
ST LOUIS MO
BLDG
31'-6"
10'-0"
32'-0"
30'-0"
CRUSHED ROCK ROAD
RIP-RAP IF REQ'D
GRASS
WATER LINE
1
3
1
2
5'-0"
INLET LINE
4'x4'x6" CONCRETE PAD
DIKE DETAIL


GRATE
℄ LOWER INLET
℄ OUTLET
℄ INLET
℄ DRAIN
GRATE
℄ GATE VALVE
4'-0"φ x ½" ...
℄ LOWER INLET
℄ OUTLET
℄ INLET
1'-6"
5'-0
℄ DRAIN
6'x6'x8" CONC. PAD
RALSTON PURINA COMPANY
ENG. DEPT.
ST. LOUIS, MO
BLDG
LAGOON OUTLET STRUCTURE


ANAEROBIC LAGOON
AEROBIC LAGOON
11 Acres
3 Ft. Deep
367 Lb. B.O.D./Day
RALSTON PURINA COMPANY
ENG. DEPT.
ST. LOUIS, MO
BLDG.

because if maintenance were ever required on this pipe it might be more difficult if it were extended out into the lagoon an excessive distance. To my knowledge we have never had a lagoon inlet line clog.

Next slide, please – This slide is the lagoon outlet structure used several times. It is fabricated from steel plate and anchored to a heavy concrete pad to prevent flotation. The main inlet is a welded tee fitting. This prevents any syphoning effect and allows water to be taken from underneath the surface so that any algae floating on the surface will be excluded. There is another inlet point, as you note, approximately 18" below the tee inlet so that the lagoon level can be lowered for maintenance purposes. When the water level is lower, of course, it is much easier to mow any weeds that may be growing on the water side. Also, I understand from some people who have rodent and animal problems in the dikes that lowering the lagoon somewhat for a period of time causes the animals to make dens lower in the dike. Then when the water level is again raised, these burrows are flooded, thus eliminating the problem. A wooden structure on piles is built out to these outlet structures so that the gate valves can be easily operated and the unit inspected periodically. A very amusing problem developed at our Trussville, Alabama plant a couple of years ago. The lagoon was about to overflow its banks. Inspection of the outlet structure showed that a large turtle had become lodged in the outlet pipe. Removal of the turtle solved the problem.

8. Next, I would like to give you a specific example of results you can expect with one of these lagoon systems. Let's take our system at Trussville, Alabama. Slide, please. There we have a rectangular clarifier for primary treatment. Thereafter, we pump the effluent from the clarifier up over a hill and into an anaerobic lagoon. From the anaerobic lagoon we flow to an 11 acre aerobic lagoon and from there to the creek. The wastes there are from a poultry dressing plant plus a rendering operation plus several hundred people in the plant. The load was tested at 11,500 BOD/day.

We arrived at this specific system because of lack of space. The original owners originally had dumped their waste into an improvised lagoon which was about 6 acres in size and 2 feet deep at the shallow end and 15 feet at the dammed end. This lagoon was not sufficient in size but did give a treatment efficiency of around 60% BOD reduction. We decided the thing to do was to install primary treatment where we could get a calculated 30% BOD reduction then flow to the anaerobic lagoon for a further 60% reduction and then to the new aerobic lagoon for a 90% reduction. Tests performed by the state indicate the following:

9. In summary, I would recommend to you in order of least total cost per unit BOD removal
    1. Anaerobic-aerobic treatment. Please keep in mind, however, that you should be out in the country far enough to be free of any odor problem. These anaerobic lagoons, while not as bad as some treatment plants around the country, do give off odors at times.
    2. Aerobic lagoons. These lagoons would vary in depth from 3 feet to 6 feet.
    3. Mechanically aerated aerobic lagoons in combination with a second cell of aerobic lagoon. Keep in mind that for maximum economy it is advisable to strive for the least horsepower possible to get the job done. This will mean striking a balance between your natural area and your aerated area. Please keep in mind that if you are short on area you can go to power aerated lagoon depths even to 30 feet if your soil conditions will allow it.

    Next, your mechanically aerated aerobic lagoons would be your most economical.

A COMPARISON OF AN EFFICIENT LAGOON SYSTEM WITH OTHER MEANS OF SEWAGE DISPOSAL IN SMALL TOWNS

by

Karl Schurr[1,2,3].

INTRODUCTION

Our nation will increase its population by one hundred million in the next thirty years. We are also developing the agricultural and industrial productivity which is continuing to provide a better standard of living for our citizens. This phenomenon of economic *and* population growth is causing a near logarithmic increase of pollutants by people, agriculture and industry (Schurr, 1970). Intensive efforts can result in a significant abatement of water pollution as described for the community at Lake Tahoe by Abelson (1970). However, a nation-wide improvement can only be effected if the needed treatment facilifites can be financed by each community. It is unrealistic to expect an average tax base in the United States to equal that of Lake Tahoe.

The present paper describes a very efficient sewage lagoon system located at Deshler, Ohio. It is compared with a well-operated primary-secondary treatment plant at Bowling Green. The initial purpose was to determine if a lagoon, added to the Bowling Green system, might be capable of enhancing the quality of the effluent. Several "borrow pits" have been dug in recent highway construction and these could form part of a lagoon system. A secondary goal was to find why Deshler has such an excellent lagoon operation.

We have made observations on the Deshler lagoons for some seven years. Various studies have been conducted on algal populations, bacterial counts and water chemistry over

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[1] Associate Professor of Biology, Bowling Green State University.

[2] I am pleased to acknowledge participation in this study by John Dohms and David Schmidt as Honors Program students in research.

[3] Grant 370071 from the Environmental Studies Center supported this research.

this time. These have been short term studies, but the general results document high quality of water treatment. Regular field trips with invertebrate zoology classes have yielded excellent collections of protozoa, aquatic insect larvae, and polyp stages of the freshwater jellyfish, *Craspedacusta sowerbyi.* Obviously, the rich aquatic fauna was also evidence of conversion of sewage into water of high quality.

DESCRIPTION OF TREATMENT SYSTEMS AND EXPERIMENTAL METHODS

Deshler has a population of about 2000 people with some light industry (an electroplating business producing the usual wastes is significant to this paper). Sewage flow may vary from slightly over 300,000 gallons to nearly a half million gallons per day. This is received into a primary lagoon of nine acres. Water is transferred to second pool of 13 acres, which has a spillway leading to a stream. The depth of both pools is slightly over one meter.

Bowling Green has a well-run sewage treatment plant with settling tank, anaerobic and aerobic units. Flow will vary with rainfall, but about 5.3 million gallons will pass through the system per day. The population of the town is on the order of 16 thousand people. There is light industry, but only a tomato canning plant is significant to sewage disposal. (The stress, resulting from industry has more potential effect on the Deshler system.) In addition, Bowling Green has a university of some 12,800 students in residence. The flow of the Bowling Green sewage treatment plant enters Poe Ditch together with nearly all of the water leaving the city.

Sample stations for water analysis were at Poe Ditch for Bowling Green, and on the outflow of the first and second ponds of the Deshler lagoon systems. This gave data on several parameters for a sample of *all* the water leaving these two communitites.

Measurements were taken each week on the following: temperature, suspended solids, turbidity, conductivity, and pH. The methods of Orland (1965) were followed for: chloride (mecuric nitrate method), oxygen (azide modification of iodometric method), and BOD. Because of portability, Hach Chemical methods were employed for nitrate, nitrite, poly and ortho phosphate (Hach, 1968). Numbers of coliform bacteria were determined by the standard tests for bacteriological examination of water (Pelczar, 1965), with presumptive, confirmed and completed tests.

Data were taken weekly for the ten weeks of April, May and the first two weeks in June. This is a crucial period for sewage lagoons in our region because organic matter may have accumulated during the winter. The increase in temperature and sunlight will promote maximum change in photosynthetic populations.

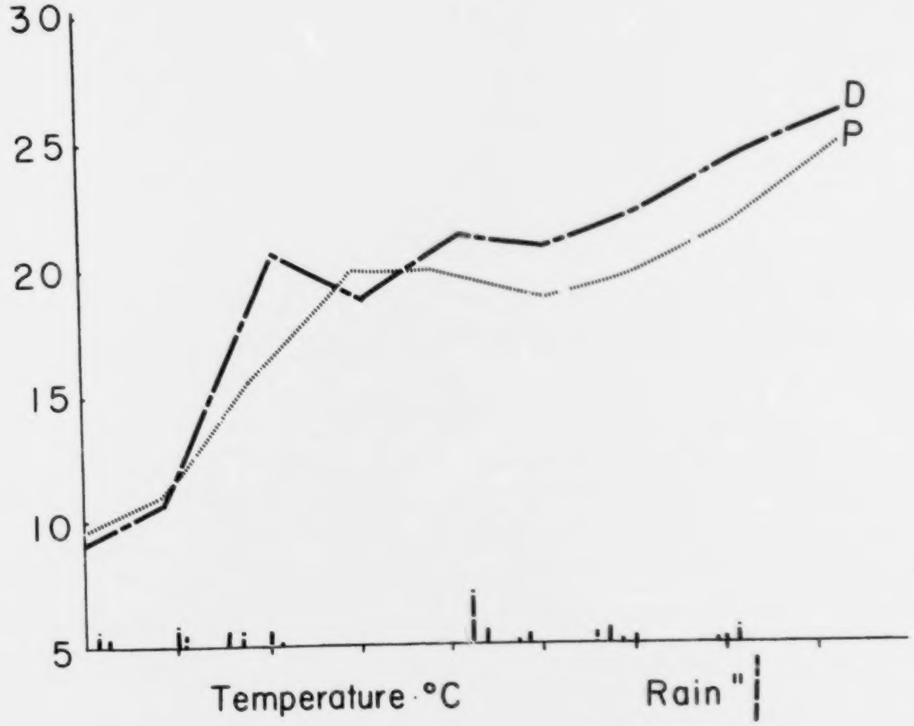

Figure 1. A graph of water tempurature during the ten weeks following March, 1970 (abscissa). Precipitation shown as a bar graph.
D = Deshler second sewage lagoon.
P = Poe ditch.

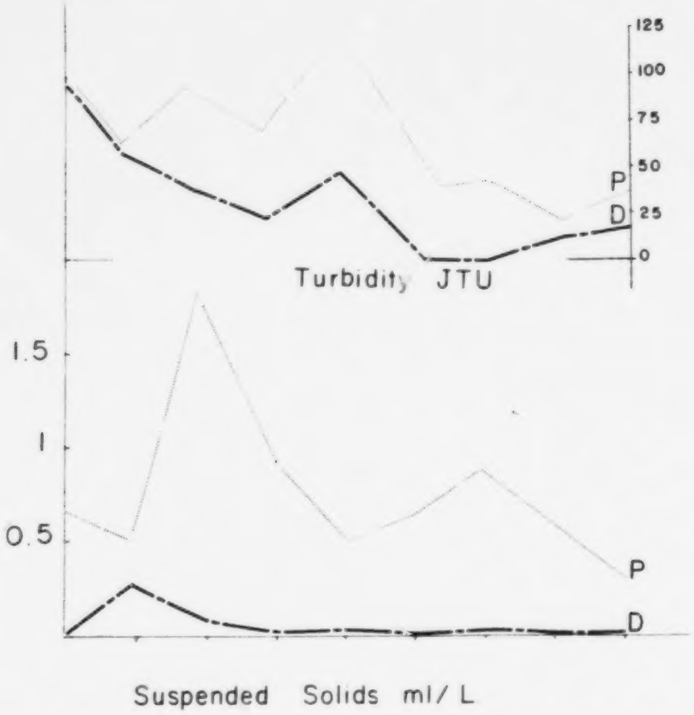

Figure 2. A graph of suspended solids during the ten weeks following March, 1970 (lower). Turbidity in Jackson Turbidity Units shown above.

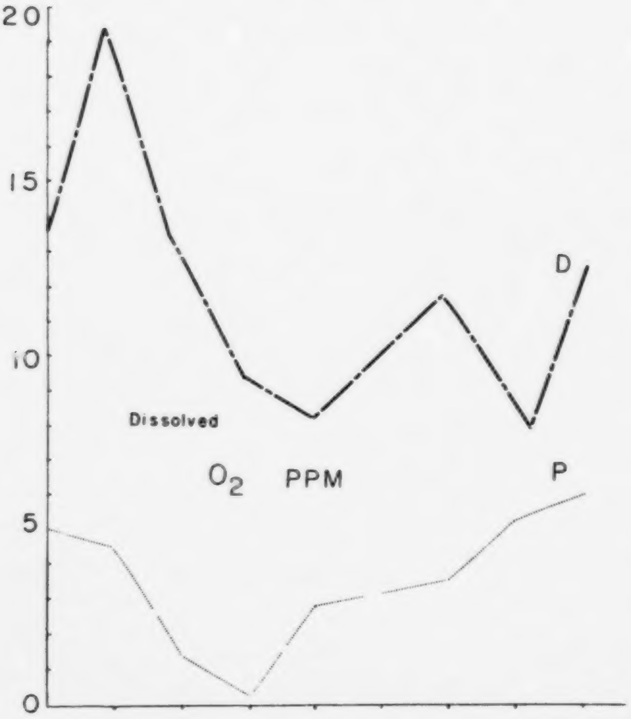

Figure 3. A graph of dissolved Oxygen during the ten weeks following March, 1970.

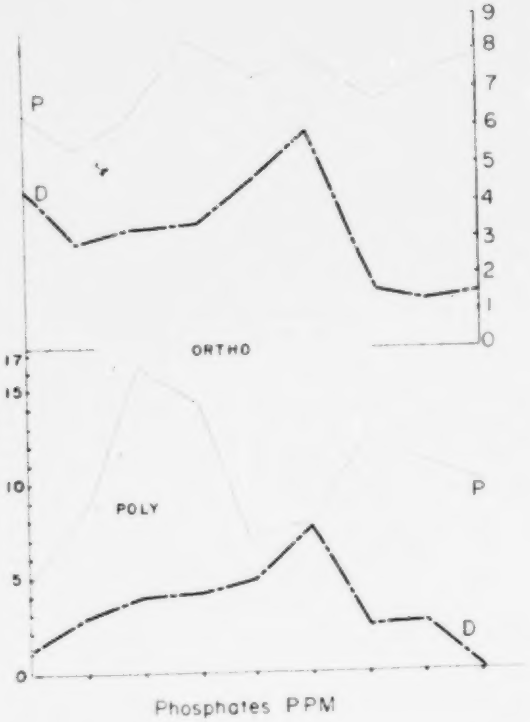

Figure 4. A graph of poly and ortho phosphates during the first ten weeks following March, 1970.

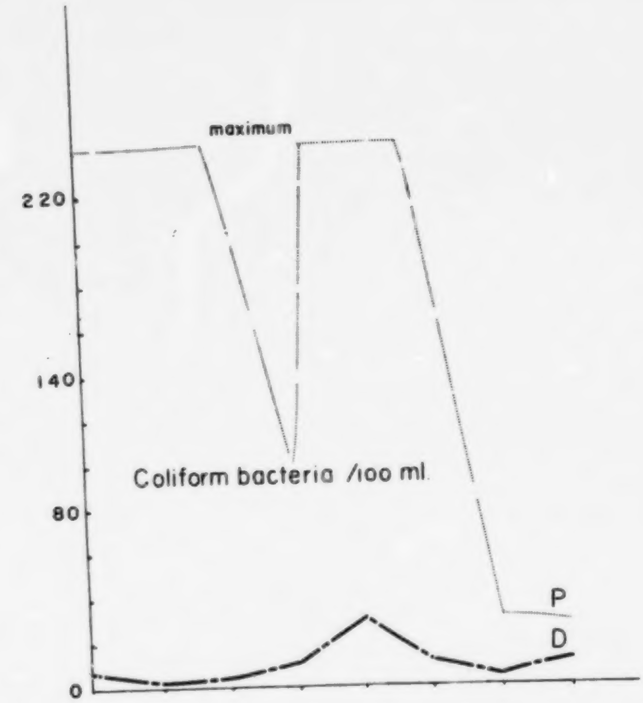

Figure 5. A graph of coliform bacteria during the first ten weeks following March, 1970. "Maximum" represents the upper limit of Most Probable Numbers of Coliforms formula used.

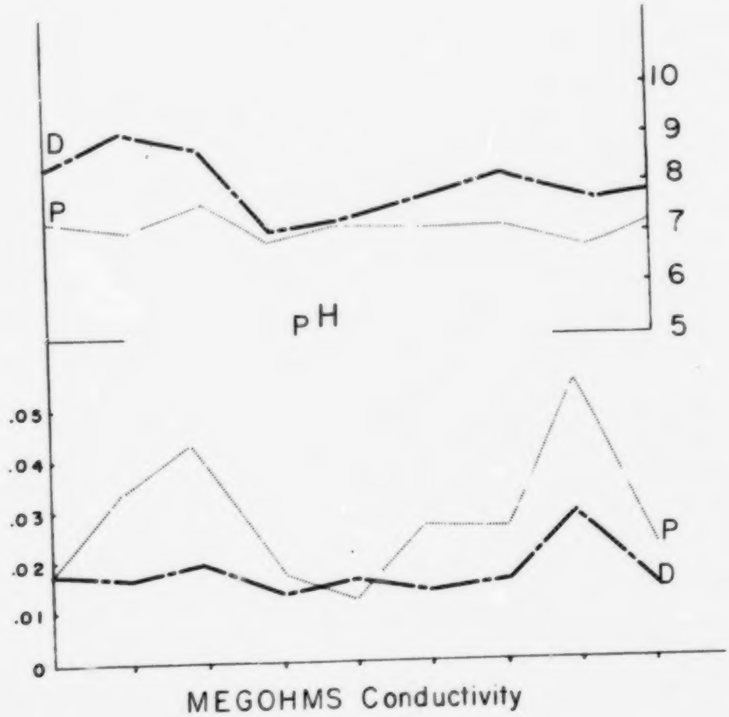

Figure 6. Graphs of pH and Conductivity during the first ten weeks following March, 1970.

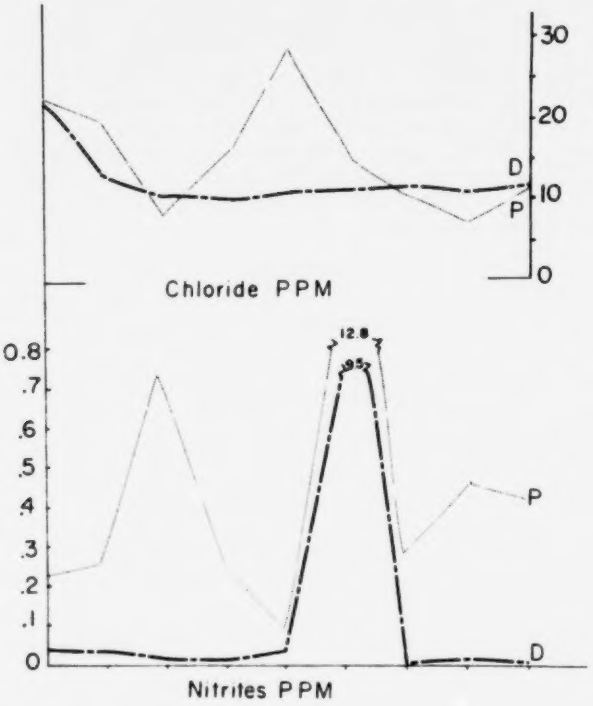

Figure 7. Graphs of Choloride and Nitrites during the first ten weeks following March, 1970.

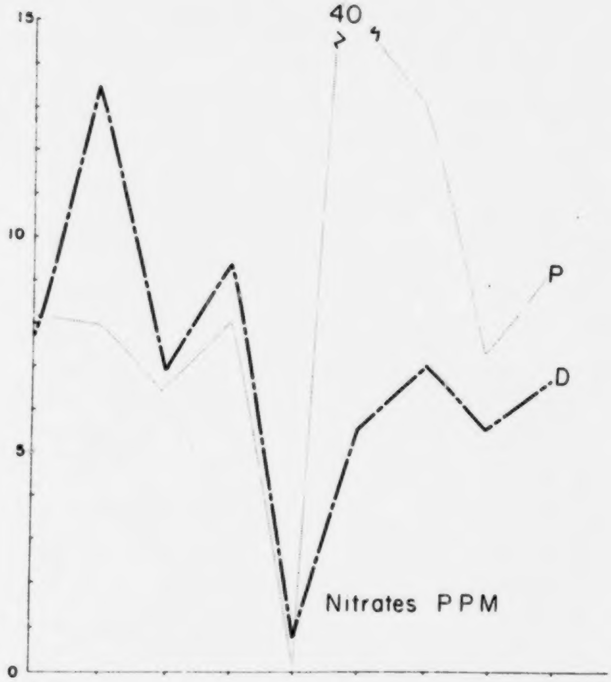

Figure 8. A graph of Nitrates during the first ten weeks following March, 1970.

RESULTS

Comparative information, on the measured parameters, is shown in the following figures. With the exception of electrical resistance, the Deshler lagoons are producing a better quality effluent. The low resistance was to be expected from the discharge of the electroplating industry.

Several of the increases documented by these data are the results of factors other than the function of the sewage treatment plant. Fertilization of University property in Bowling Green, and general fertilization of lawns in both towns, was followed by significant rainfall. This is reflected in the water chemistry. It should be emphasized that this comparison was made in order to document the quality of the Deshler lagoon treatment. No adverse judgement is intended toward the Bowling Green treatment plant.

Two possibly significant factors may promote the efficiency of the Deshler lagoons. Both of these have been described by Verduin (1970). One is the dissolved limestone in the Deshler water supply. This provides titratable base of 3 to 10 meq/liter with abundant CO_2 from such a buffer system as follows:

$$CO_2 + H_2O \longleftrightarrow H_2CO_3$$

$$Na_2CO_3 + H_2CO_3 \longleftrightarrow 2NaHCO_3.$$

An obvious test of this hypothesis would be to add titratable base as limestone ($CaCO_3$) or soda ash (Na_2CO_3) to sewage in a lagoon system which is not operating as well as expected. If there are no other deleterious factors, this treatment should result in better photosynthetic conversion of sewage to water. A second very interesting finding of Verduin (1970) was the documentation of a ten fold higher rate of CO_2 fixation, in photosynthesis, as compared to O_2 production in the Deshler Lagoon. *Scenedesmus* was the dominant phytoplankton.

Unusual CO_2 to O_2 ratios are not unknown for certain photosynthesis reactions, but in the case a detailed study should be made to find if the algal populations are truly unusual or more efficient. The data were gathered by Dr. Verduin with methods described by Verduin (1960) and also with radioactively labeled CO_2 used to document fixation.

Biochemical Oxygen Demand was measured for both the Deshler lagoons and Poe Ditch. Our data compare with that taken by the Supervisor of the Deshler system, and a typical set of readings would show BOD to be: 11 ppm for the final lagoon, 10 ppm for the receiving stream above the effluent outlet, and 10 ppm for the stream below the outlet. BOD for Poe Ditch averaged somewhat higher than the Deshler effluent.

DISCUSSION

Midwestern United States is characterized by high human population both in urban centers and also in many small towns and villages. A lagoon sewage system has limitations for a city or for most industrial operations. On the other hand, there are specific advantages for small towns: construction cost is low, land is generally available, there are minimal operational costs, and labor required for supervision and control is limited.

The unfortunate situation in the Midwest (and in other geographic regions of our country) is the lack of any municipal sewage treatment in a great number of the small towns. A septic tank, with filter bed, can be an effective method for an individual home. The difficulty lies with shoddy construction, or with improper maintenance. It is common to find a shortcut of the drain line into field tiles. Other ploys, by get–rich–quick builders, may be used. Perhaps the most interesting case of this sort in the writer's experience was the individual who was brought to court because of a suspected violation of a local code. He made the statement that a particular family had *no waste water!*

The majority of small towns have exceeded population levels where individual home treatment of sewage is safe for public health. Many contribute significantly to pollution in the surface and ground water. The future will certainly bring a great increase in legally mandated changes in sewage treatment in order to prevent epidemics. I suggest that an efficient lagoon, such as the Deshler system, is the best alternative for most villages and small towns. Furthermore, a lagoon system may be a logical alternative for somewhat larger towns in which population growth has overloaded a standard sewage treatment plant. In this case, lagoons might be used to receive excessive flow resulting from rain. On the other hand, the best method might be to channel all effluent through a lagoon system after the usual anaerobic and aerobic treatment.

Considerable research should be done on the effects of pH, titratable base, and possible unusual photosynthetic rates as mentioned previously. I suspect that unsatisfactory lagoon systems might be made efficient with manipulation of one or more of these factors. Other research should be initiated on aquaculture of fish and other animals in efficient lagoons. (The potential for these benefits is described by Ryther, 1968 and Bardach, 1968.) Tests of algae as food supplement for domestic animals are an obvious extension of lagoon research. For example, ensilage might receive fortification with algae and make a better food for cattle.

The Department of Biology at Bowling Green State University is fortunate to have the faculty, the facilities and such treatment systems operating nearby; we expect to be able to continue research on these problems with appropriate funding. The major goal, however, is to determine if lagoons can be made more efficient and to assess the utility of adding a lagoon system in towns where the standard sewage treatment plant is overloaded because of population growth

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CARBON BUDGET AND TOTAL PRODUCTIVITY OF AN ESTUARINE OXIDATION POND RECEIVING SECONDARY SEWAGE EFFLUENT

by

John W. Day, Jr., Charles M. Weiss and H.T. Odum
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INTRODUCTION

Secondary sewage treatment plant effluents are flowing with increasing frequences into the estuaries of our coastal regions. National planning now projects many more treatment plants. What are the consequences of these increased flows? What kinds of ecological systems will develop under regular regimes of sewage effluents mixing with salt water? Can the ecosystems that develop at the interface between man and nature process and recycle minerals in a compatible way with marine resources? Can such systems yield food products? To aid in answering some of these problems an interdisciplinary project is currently under way to test the self designing principle by which a stable and seasonally repeating estuarine ecological system could be made compatable with treated sewage.

A major part of this project is the determination of the productivity and carbon flux of a pond system, shallow basins receiving treated sewage effluent and mixed to a constant salinity with estuarine water. Formulation of a model of the pond system will provide a basis for understanding of the pond, and more broadly, of the estuarine ecosystem. If this model is a valid expression of this type of system, it can be used in predicting the consequences of environmental changes. Manipulation and measurement using the model can be accomplished at a considerable saving in time, effort and danger to the system as compared with equivalent manipulations of the real system. In general, both the pond experiment and the mathematical expressions are simulations of a pollution-stressed estuarine system.

The overall metabolism of aquatic ecosystems is readily monitored by the diurnal changes of substances that are produced by photosynthesis and respiration. Montoring of oxygen and carbon dioxide changes for metabolic studies have been frequently used. (Odum and Hoskin, 1958; Hoskin, 1959; Odum and Wilson, 1962; and Copeland and Dorris, 1964.)

DESCRIPTION OF THE PONDS

Six ponds approximately 60 x 100 feet with an average depth of 18 inches were constructed. Three control ponds (C-ponds) are located on the grounds of the Institute of Marine Sciences at Morehead City, N. C., the three waste receiving ponds (P-ponds) are nearby at the Morehead City sewage treatment plant on Calico Creek (Fig. 1). After the basins for the ponds were scooped out, a 2-3 feet clay seal lined both sets of ponds. Then the bottoms were floored with about a foot of black marsh mud that had been excavated from the Calico Creek area. A standpipe was set in the middle of each pond at a depth of 3-4 feet. A small pier extending to the drain pipe in each pond was fitted with an instrument shelter supplied with electricity.

The control ponds receive a mixture of tap water and estuarine water from Bogue Sound. The waste ponds receive treated sewage and estuarine water from Calico Creek. This water was much enriched by the main treated sewage discharge nearby. The flows in the ponds were adjusted so that a salinity between 15 and 20 parts per thousand was maintained with an average retention time of approximately 30 days.

To facilitate establishment of a compatable ecosystem extensive seeding of the ponds took place in the spring of 1969. Plankton collected in night tows from the nearby Sounds were used. Additional plankton and larval stages of larger organisms were continually introduced by the salt water pumps. Because of this seeding a fairly high diversity rapidly developed in both sets of ponds.

Water began flowing into the ponds in September, 1968. Productivity studies were started in the fall of 1968. Studies of the carbon flux in the ponds began in May, 1969.

METHODS

When simultaneous oxygen studies were made on all six ponds, diurnal records were made with Winkler analyses at 3 hour intervals. Continuous records were made with an oxygen electrode. Winkler analyses were used at intervals to check the electrodes. For daily total oxygen production it was only necessary to take oxygen measurements at the point of minimum and maximum on the diurnal curve. Over much of the study, the instrumental recorders were moved from one pond to another each week, rotating so as to cover them all once every six weeks. A computer program written in PL/1 language was used to accept diurnal data and plot graphs of oxygen, percent saturation, temperature, and rate of oxygen change.

Light was recorded with an integrating photometer manufactured by the DTI Division of Talley Industries, Inc. The insolation received was read by reading the ampere-hour meter at dawn and dusk, substracting the readings and multiplying by the factory calibration: Amp Hrs x 177 = cal/cm^2. The insolation meter was read each day during most of the year. If net daily oxygen gain (oxygen production) is given a rough energy equivalent of 4 kcal/g it may be compared with insolation as efficiency of production with both imput of sun energy and output of oxygen being in kcal/m^2/day.

A continuous temperature record was made with a thermistor sensor. In very shallow ponds the temperature variation diurnally and annually is a factor of major interest in understanding the ecosystems which develop.

Records of pH were made with a Fisher pH meter with long leads, the electrodes hanging in the water in the center of the ponds. Calibration of the electrodes were made once or twice weekly.

In determining the carbon flux in the ponds, several measurements were made. The mixing tanks and the ponds were sampled to determine the carbon content of the influent water and the ponds. The flow rate of the entry and drain were measured so that the carbon flux in the aqueous phase could be calculated. Carbon dioxide diffusion across the air-water interface was measured with an infrared gas analyzer.

Samples were collected weekly from each pond and mixing tank and preserved with 40 mg/l Hg^{++} so that no biological degradation of organic material would occur. The pH was measured at the time of collection. This value was designated the field pH (FpH). The carbonate alkalinity of each sample was determined by titration. Because of irregular pumping intervals at the P-pond mixing tank, it was felt that weekly samples might not be representative. Therefore, a method was devised to collect a sample which was integrated over time. A sampling peristallic pump was installed that pumped a constant low flow whenever the sewage or salt water pumps were operative. This flow was collected in a five gallon bottle from which a subsample was taken when full. Hg^{++} was used as a preservative to minimize biological activity during the collection period.

The carbon content of the water samples was analyzed with a Beckman total carbon analyzer. (Williams, 1966.) The total carbon (TClab) was determined as follows. A 40 Ml sample was injected into a stream of CO_2-free air which was carried into a high temperature furnace where it was combusted to carbon dioxide. The carbon dioxide was then swept into an infrared gas analyzer.

Corrections for CO_2 diffusion have to be made to obtain the total carbon of the sample at the time it was collected. The pH was measured at the time of analysis and designated Lab pH (LpH.) The TC_{lab} was read from the total carbon analyzer read-out. The total inorganic carbon (TIC_{lab}) at the time of analysis was calculated as follows:

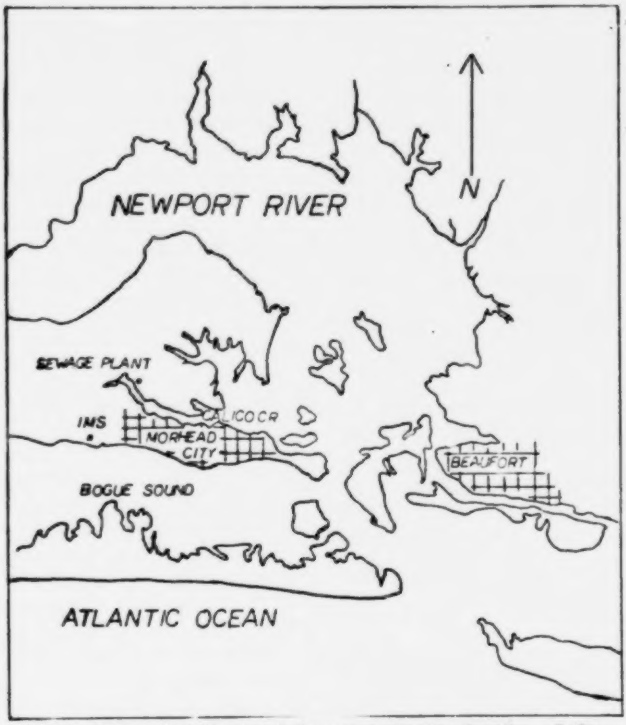


Figure 1. Beaufort-Morehead City area showing location of the Institute of Marine Sciences (IMS) and the sewage treatment plant.

$$\text{Alkalinity} = (A1 + 2A2)\ TIC + OH^- + H^+$$

where

$$A1 = \frac{1}{1 + \frac{H^+}{K_1} + \frac{K_2}{H^+}}$$

$$A2 = \frac{1}{1 + \frac{H^+}{K_2} + \frac{H^{+2}}{K_1 K_2}}$$

(Fair, Geyer, and Okun, 1966)

These are proportionally constants which give the fraction of the different species of inorganic carbon at varying pH values. OH^- and H^+ are negligible at normal pH values of natural waters. The values for the equilibrium constants were obtained from Riley and Skirrow (1965). Rearranging, the following is obtained.

$$TIClab\ (mg/l) = (\text{alkalinity, mM/l})\ (12\ mg/mM)\ /(A1 + 2A2)$$

The LpH is used in determining values for A1 and A2. The total inorganic carbon of the sample at the time it was collected (TIC_{field}) is calculated using the same formula except the FpH is used in calculating values for A1 and A2. To obtain the total organic carbon (TOC), inorganic carbon is subtracted from total carbon.

$$TOC = TC_{lab} - TIC_{lab}$$

This is the Total Organic Carbon in the sample at the time of collection since it was preserved with Hg^{++}.

$$TOC + TIC_{field} = TC_{field}$$

This is the total carbon of the sample at the time it was collected.

The values for carbon include carbon of microscopic plankton organisms and detritus but not that of larger organisms such as fish or of such material as blades of grass. At regular intervals, a cloth mesh was placed over the drain to catch any large material flowing down the drain. The amount of carbon lost through this pathway was found to be insignificant when compared with that in the water samples.

Carbon dioxide diffusion was measured with the apparatus shown in Fig. 2. The air was sampled before

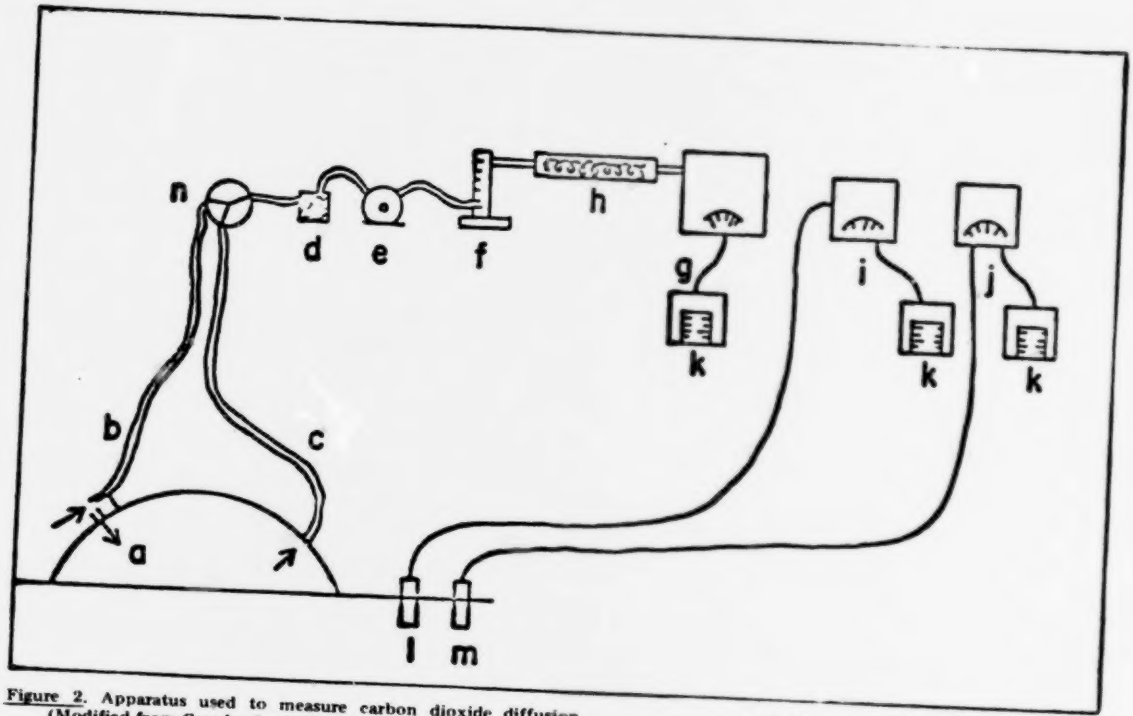


Figure 2. Apparatus used to measure carbon dioxide diffusion (Modified from Copeland and Duffer, 1964).

a. floating plastic dome
b. ambient air tube
c. dome air tube
d. drierite water trap
e. pump
f. flowmeter
g. Beckman 215A infared gas analyzer
h. glass wool filter
i. pH meter
j. temperature meter
k. recorders
l. pH electrodes
m. thermister
n. solenoid valve with timer

(tube a) and after (tube b) as it flowed under a 40.5 cm dome floating on the surface of the pond. A solenoid valve fixed with a rotating cam timer was operated to sample the air for 15 minutes before entering and for 45 minutes after it had passed under the dome. The air then flowed successively through a water trap, pump, flow-meter, and glass wool filter. The flow through the system was adjusted to 1.8 1/min. Several trials were conducted at flows ranging from 0.5 to 2.0 1/min. Flow rates greater than 1.5 1/min. had no effect on the rate of carbon dioxide diffusion. The air then flowed into a Beckman 215A infrared gas analyzer to measure carbon dioxide concentration. Continuous recordings were made of pH and temperature. The rate of diffusion was calculated as follows (Odum,1970)

$$\frac{(CO_2\text{ dome} - CO_2\text{air, ppm})}{(106)} \times$$

$$\frac{\text{flow 1}}{\text{min}} \times \frac{60\text{ min}}{\text{hour}} \times \frac{(1\text{ mole}}{22.41} \times \frac{12g)}{\text{mole}} \times$$

$$\frac{(273)}{(K^o)} \times \frac{(\ 1\ )}{(\text{area m}^2}$$

The rate has the units of g C/m^2/hr. The concentration of CO_2 in the water (CO_{2w}) is calculated form alkalinity data.

$$\text{TIC mM/l} = (\text{Alk, mM/l} / (A1 + 2A2)$$

and $$CO_{2w} = AO \times TIC$$

where

$$AO = \frac{1}{1 + \frac{K_1}{H^+} + \frac{K_1K_2}{(H^+)^2}}$$

AO is the fraction of the inorganic carbon which is dissolved carbon dioxide. The carbon dioxide concentration of the air is calculated in the same units.

$$CO_{2a} = P_{CO_2} \quad CO_{2a} = P_{CO_2} \times a$$

where :

P_{CO_2} = partial pressure of CO_2 in air in atmospheres

a = Henry's law coef. in mM/l atm (Riley and Skirrow, 1965)

CO_{2a} = conc of CO_2 in air in mM/l

The rate of diffusion (R) of carbon dioxide in proportional to the gradient of partial pressure of CO_2. If the partial pressure is greater in the water, CO_2 will diffuse out; if the partial pressure is greater in the air; it will diffuse in. If R is proportional to the gradient then R = k gradient. Thus k = $R/((CO_{2a} - CO_{2w})$. This is a modification of the Bohr equation used in studies of gas-liquid diffusion (Skirrow, 1965). The units of k are

$$\frac{g\ C/m^2/hr}{mM/l\ CO_2}:$$

The dome method for measuring CO_2 diffusion can be used if one assumes that the limiting factor in the diffusion is transport of dissolved CO_2. Bolin (1960) indicates that it is slow transport in the aqueous boundry layer which is the rate determining step. Hanya and Ishiwatari (1961) found that the diffusion constant tended to increase with increasing turbulence. This is consistent with the general tendency for the constant to increase at night, presumably because of the increased turbulence due to turning over of the ponds.

RESULTS

Diurnal and annual changes in the experimental variables are presented in Figs. 3-13.

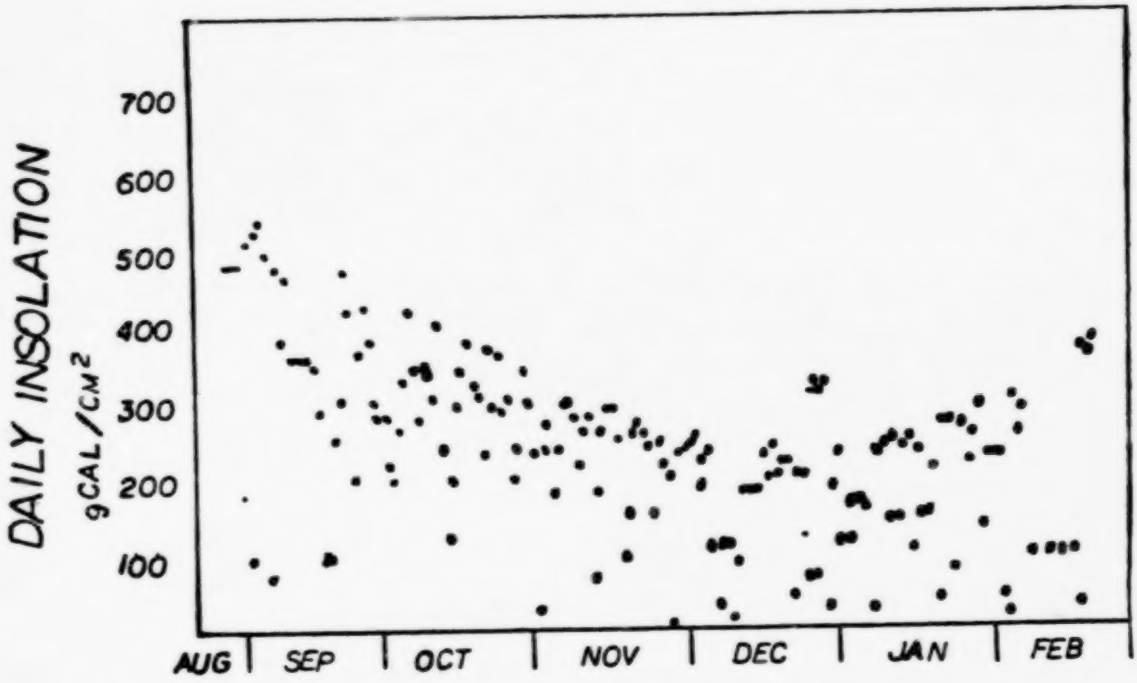


Figure 3. Daily and seasonal variation in insolation. Low values indicate cloud cover.

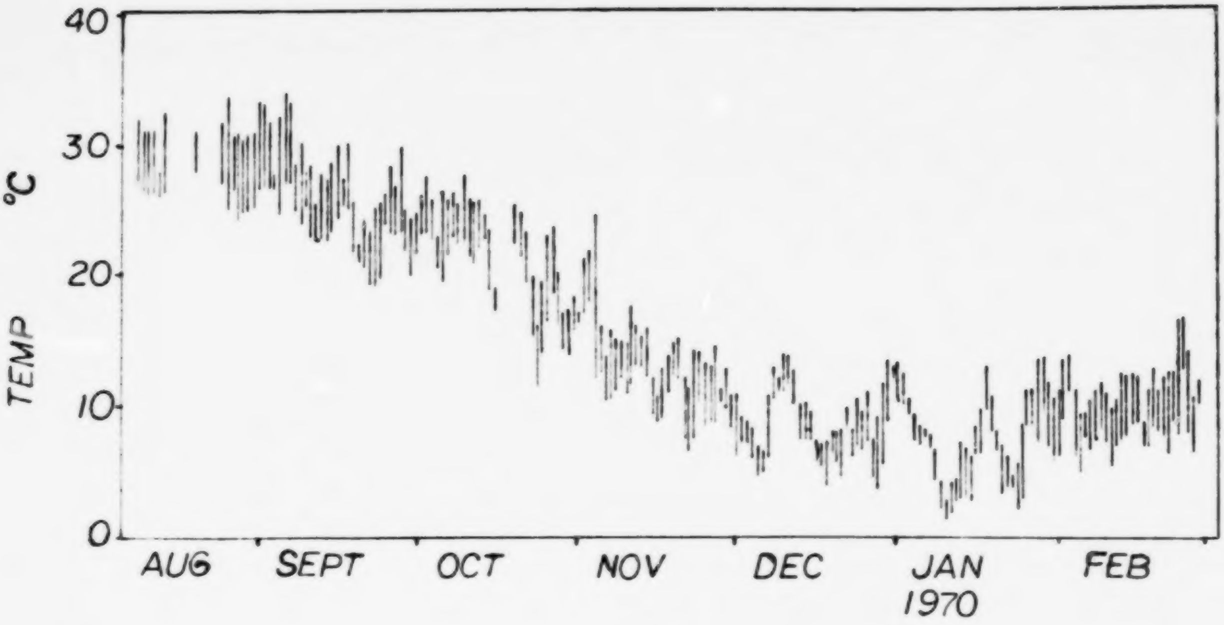


Figure 4. Diurnal and seasonal variations in temperature.

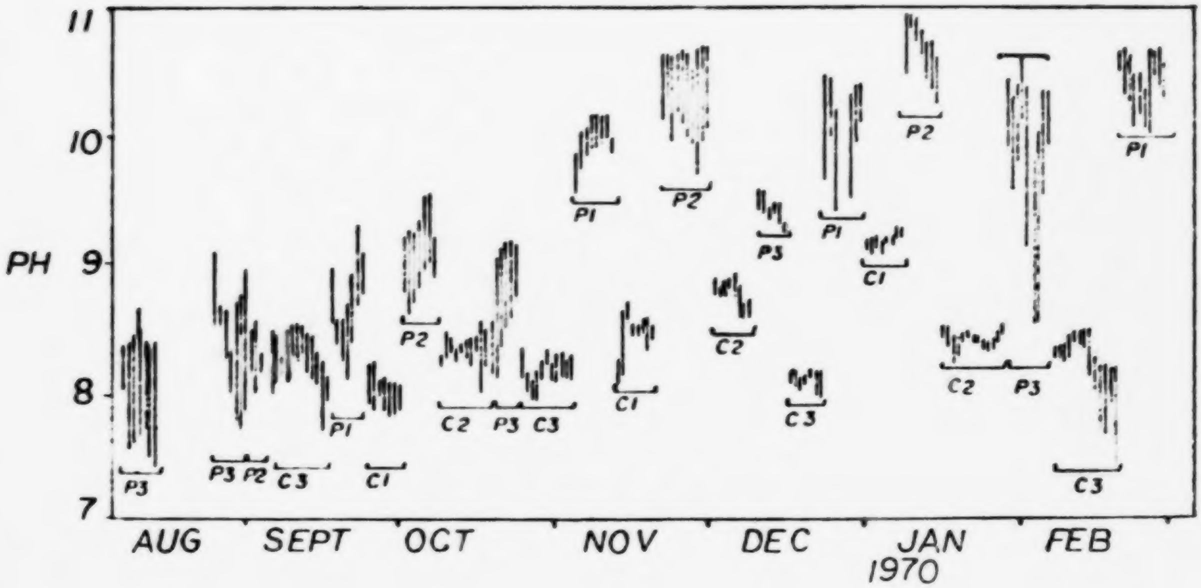


Figure 5. Daily and seasonal pH variations in various ponds. High values result from algal blooms.

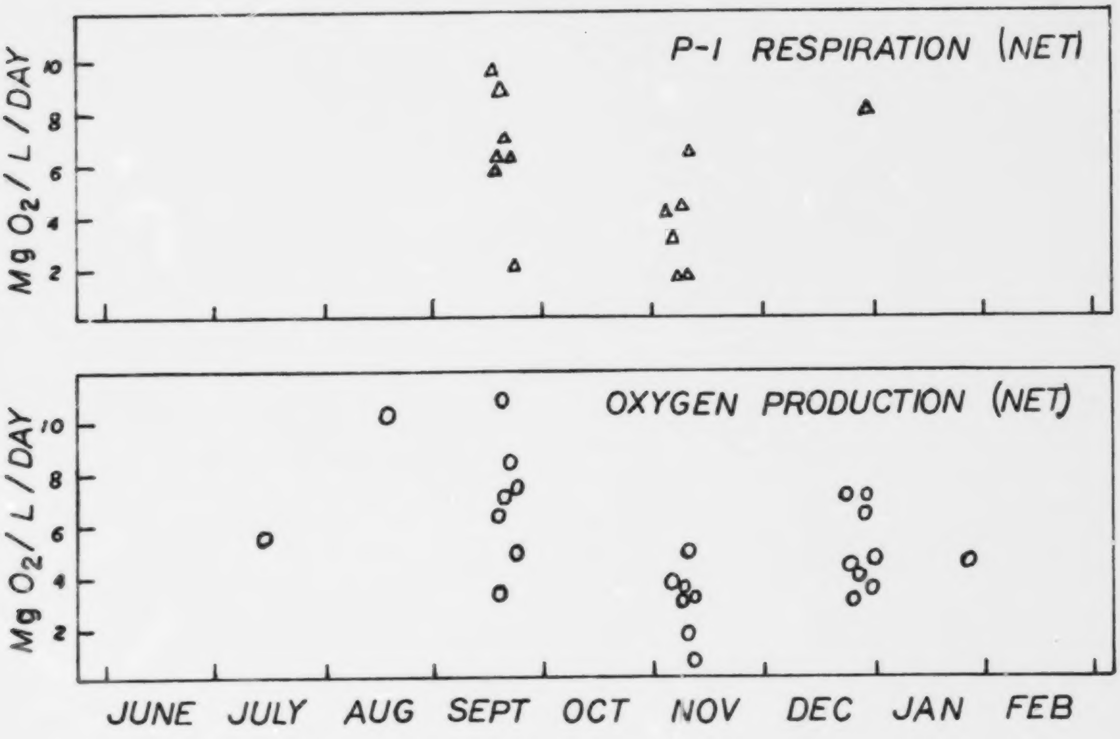


Figure 6. Net respiration and oxygen production for a representative waste receiving pond.

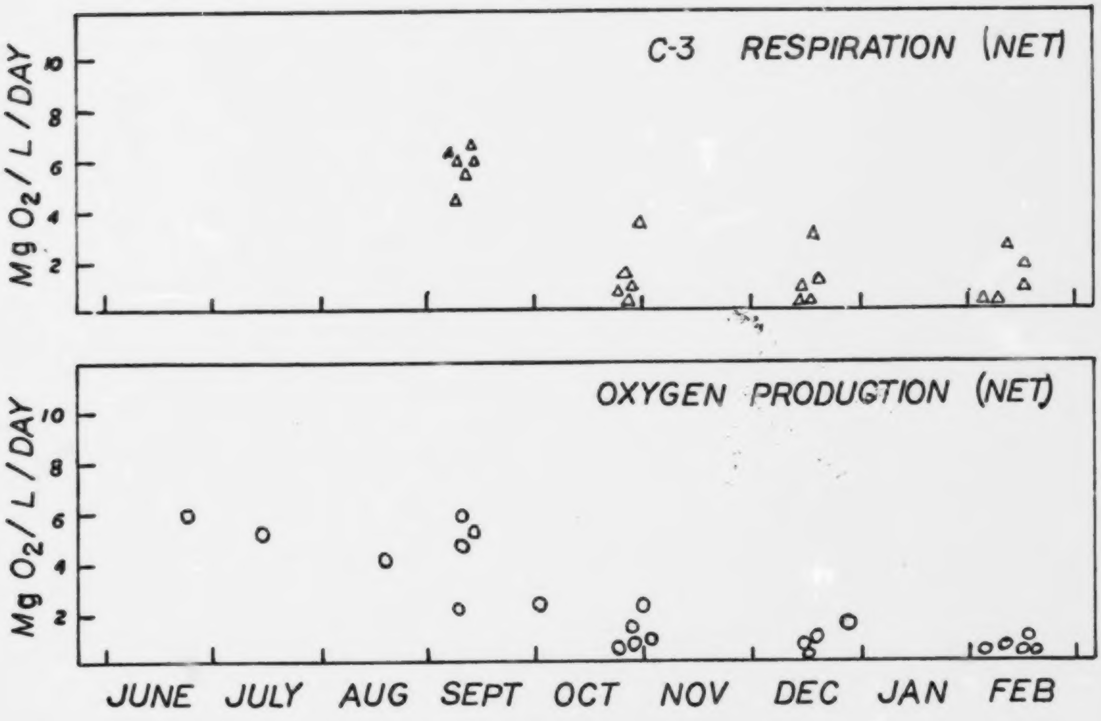


Figure 7. Net respiration and oxygen production for a representative control pond.

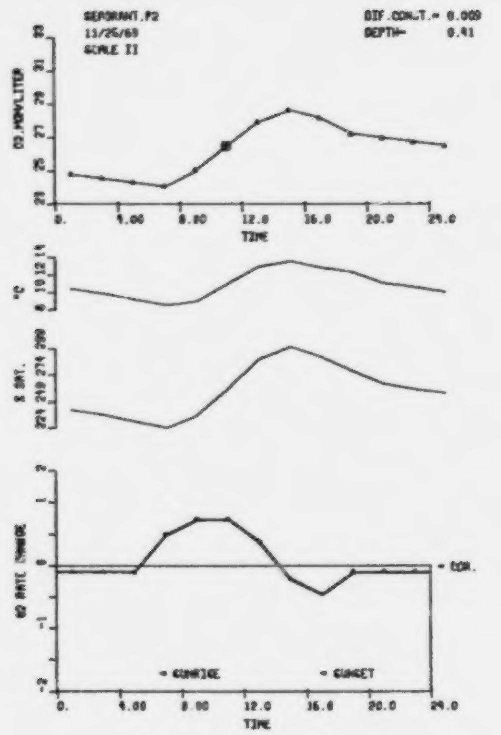


Figure 8. Print out from oxygen program showing diurnal oxygen, temperature, percent saturation, and rate of oxygen change.

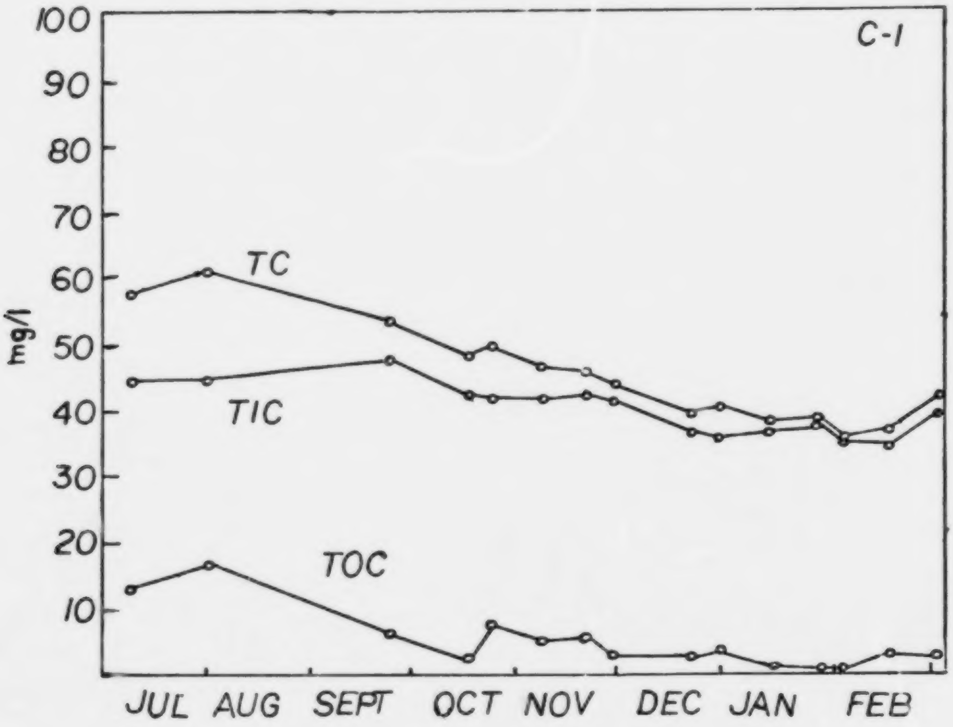


Figure 9. Seasonal variation in carbon levels in a representative control pond. TC, total carbon; TIC, total inorganic carbon; TOC, total organic carbon.

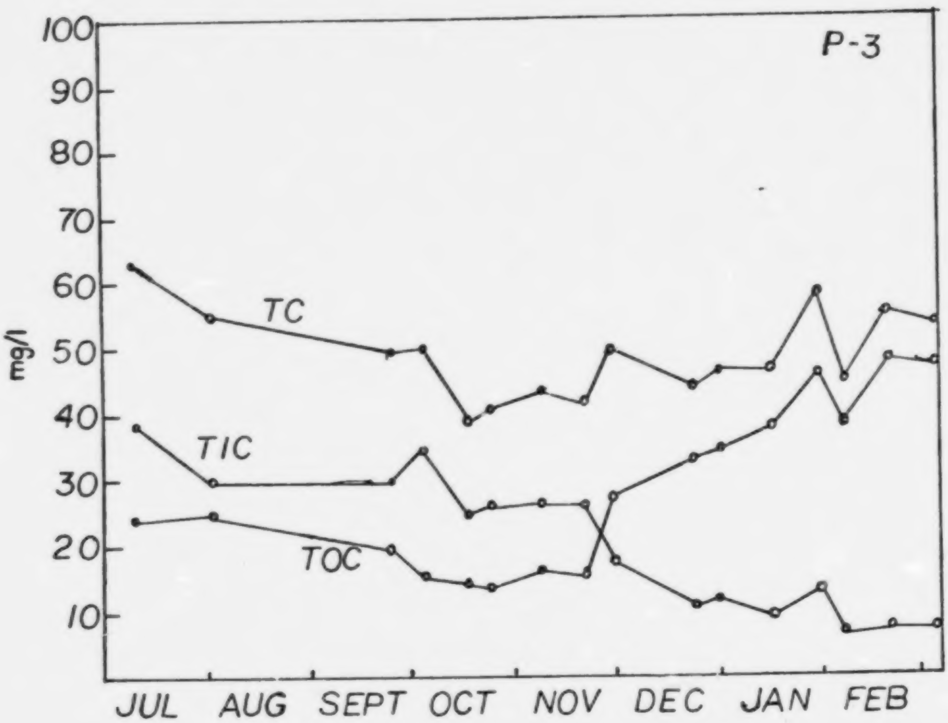


Figure 10. Seasonal variations in carbon levels in a representative waste receiving pond. High organic levels beginning in November are resultant from algal bloom. TC, total carbon; TIC, total inorganic carbon; TOC, total organic carbon.

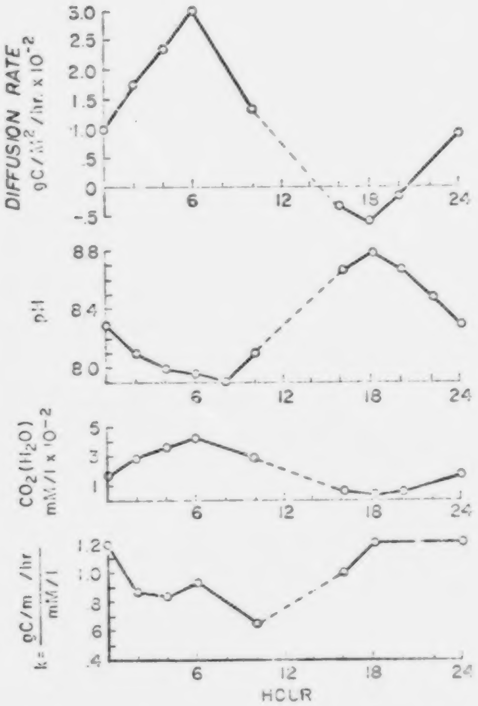


Figure 11. Diffusion rate, pH, dissolved CO_2, and diffusion constant for a representative waste receiving pond before bloom conditions.

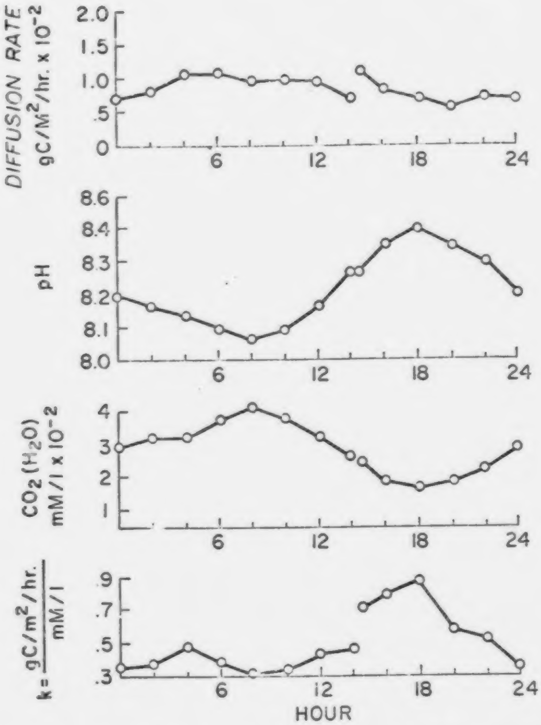


Figure 12. Diffusion rate, pH, dissolved CO_2, and diffusion constant for a representative control pond.

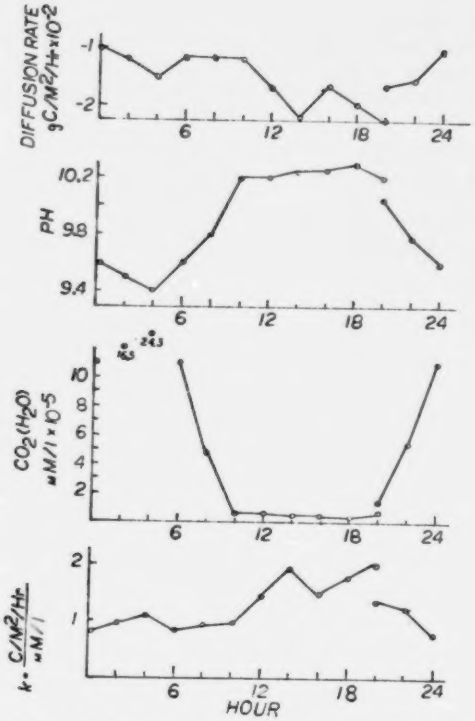


Figure 13. Diffusion rate, pH, dissolved CO_2, and diffusion constant for a representative waste receiving pond during bloom condition.

Insolation data for each day are given in Fig. 3. The sharp changes with clear and cloudy weather impose a three fold range over the shifting energies expected with season due to high summer sun angle and long day length at this location, 35 deg. N. latitude.

The continuous temperature record shows that the pond temperature follows the weather conditions with sharply changing magnitudes in winter. Daily ranges are shown in the figure and these are largest when insolation is greatest. The temperature range serves somewhat as a net radiometer for total insolation received and correlates with insolation data.

Oxygen and pH levels have the expected sharp diurnal range, being the greatest during light conditions of summer and varying with algal blooms. Seasonal values for pH along with daily ranges are shown in Figure 5. The extremely high pH values in the waste ponds from November through February were due to bloom conditions. Representative values for photosynthesis and respiration are given in Figs. 6 and 7. During periods when photosynthesis was exceeding respiration and diffusion was relatively unable to buffer the interior gases with outside air, the oxygen and the pH rose to extreme values, oxygen was supersaturated and pH values as high as 10 were recorded in the fall, 1969. At other times there were very low DO and pH values as respiration exceeded photosynthesis for several days. With a continual outflow, removing organic matter, there was an overall tendency through the year for high oxygen and pH to be maintained, the ponds being of the net production type. Low oxygen was never a general property. Figure 8 shows a typical diurnal record of oxygen diffusion obtained from the oxygen program.

Figures 9 and 10 shows the levels of total, organic and inorganic carbon from July, 1969 through March, 1970. Figure 10 gives the carbon levels in one of the ponds receiving treated sewage. This record is typical of the results from the polluted ponds. From mid summer until November the organic carbon levels were below that of the inorganic levels. In the latter part of November, an agal bloom developed on all of the P-ponds. This bloom persisted past the time when the last measurements were taken. This is shown in the graphs where the organic carbon levels went from an average of 15 mg/1 to between 40 and 50 mg/1.

The concentration of organic carbon in the effluent from the trickling filter averaged 15 mg/1. This is normal for units of this type. Rich (1963) reports that the levels of organic material from such filters is usually less than 20 mg/1. The levels from the trickling filter serving Chapel Hill, North Carolina vary from 12 to 25 mg/1 (Lamb, 1970.)

Figure 9 shows the record for one of the control ponds for the same period. The organic carbon remained below 10 mg/1 for most of the sampling periods, several being undetectable. The water in the control ponds was typically clear and approximated Bogue Sound water.

Figures 11 through 13 present data on carbon dioxide diffusion. Normally carbon dioxide would diffuse into the water during light hours due to uptake during photosynthesis and out at night due to respiratory production. Positive values for the diffusion rate indicate diffusion out and negative values are diffusion into the water. This pattern is evidenced in P-3 as shown in Figure 11. However, in the control ponds carbon dioxide diffused out (Fig. 12). This was due to the high inorganic carbon level in the city water used to dilute the C-ponds. It was obtained from wells in a limestone aquifer and was supersaturated with inorganic carbon when it reached the ponds and CO_2 continually diffused out.

Parks (1969) found that the water at the mouth of the Columbia River was supersaturated with inorganic carbon and that carbon dioxide diffused out. This is consistant with the constant outward diffusion of the C-ponds. The diffusion curves have the normal diurnal shape, but the whole curve is shifted upward on the graph.

After the continuous blooms began in the P-ponds, the diffusion pattern changed (Fig. 13). The rate of photosynthesis was high enough so that the partial pressure of CO_2 was always lower than that of the atmosphere. Dissolved CO_2 is low during the day due to photosynthetic uptake resulting in higher pH values. The CO_2 level rises at night due to respiration thus lowering the pH. The lower graph in each figure is the variation of the diffusion constant over a 24 hour period. Although considerable variation occurred, the general pattern was for the diffusion constant to be lower during the day. The greater mixing during the night is probably due to the fact that shallow ponds such as these turn over each day.

Rate constants for CO_2 diffusion as reported in the literature vary considerably due to the influence of turbulence. Sugiura, *et al.* (1963) reported values ranging from .072 to 23.89 g C/m2/hr/mM/1.

DISCUSSION

In general the rise and fall of oxygen and pH was that expected with normal alternation of net photosynthesis and respiration, the latter increasing during the day with storage and decreasing at night. With insolation decreasing after noon and respiration still increasing, oxygen levels often started down before sunset but not so sharply as in many ecosystems which are more stressed.

In the annual record of pH (Fig. 5) fairly normal values were present at first, but the very high pH became the characteristic pattern by Fall, with onset of lower temperatures.

The levels of net daytime photosynthesis and nighttime respiration are given in Figures 6 and 7. In general the waste receiving ponds show high net production compared to the control ponds. Respiration is also higher following on the high day light photosynthesis. In the course of the year the values of photosynthesis have generally been rising compared to what they were at the same time in the previous year. In a sense this is a form of succession, a power selection of a system capable of more metabolic process than the one it succeeded. It represents adaptation. It is not yet possible to indicate whether there is already some approach to climax or whether patterns will be fluctuating over this range in the future.

Compared with other marine pond studies (Odum and Wilson, 1962) these productivities are high. To compare these daytime net storages with the gross production data used in the Texas work one may add values of net daytime photosynthesis to those of night respiration, a procedure which usually doubles the value. Making these comparisons indicates that the Morehead City waste ponds had 2 to 10 times or more the ordinary production observed in the control ponds.

Efficiency of net production for the waste ponds was about 0.45 percent of total insolation received while that of the control ponds was 0.21 percant of total insolation received. These ponds have the same levels of efficiency as freshwater oxidation ponds. Estimating from Figures 6 and 7 the total pond net daytime photosynthesis in 1969 was 994 g oxygen/m^2/year for the waste ponds and 446 g/m^2/year for the control ponds. The total night respiration was 864 g/m^2/year for the waste ponds and 348 g/m^2/year for the control ponds.

MODELING

The model of carbon metabolism of the pond system is a digital model written in PL/1 computer language. In constructing such a model one must take into consideration the different reservoirs of carbon and the flow of carbon between these reservoirs. The storages considered are the input water, the pond, the sediment, and the atmosphere. Figure 14 is a diagram of the pond system showing the various storages and flows of carbon. The flow from any reservoir can be represented as a first order rate, with the flow being equal to the concentration on the reservoir times a transfer constant (k). The individual rates shown in Figure 14 are computed as follows:

Organic input $= K_1 x\, CO_{inp}$

Inorganic $= K_1 x\, CI_{inp}$

Respiration $= K_3 x\, CO_{pond}$

Photosynthesis $= CI_{pond} \dfrac{k_4 x L}{k_7 + L}$

Inorganic drain $= k_5 x\, CI_{pond}$

Organic drain $= k_5 x\, CO_{pond}$

CO_2Diffusion $= k_6 (CO2_a - CO2_w)$

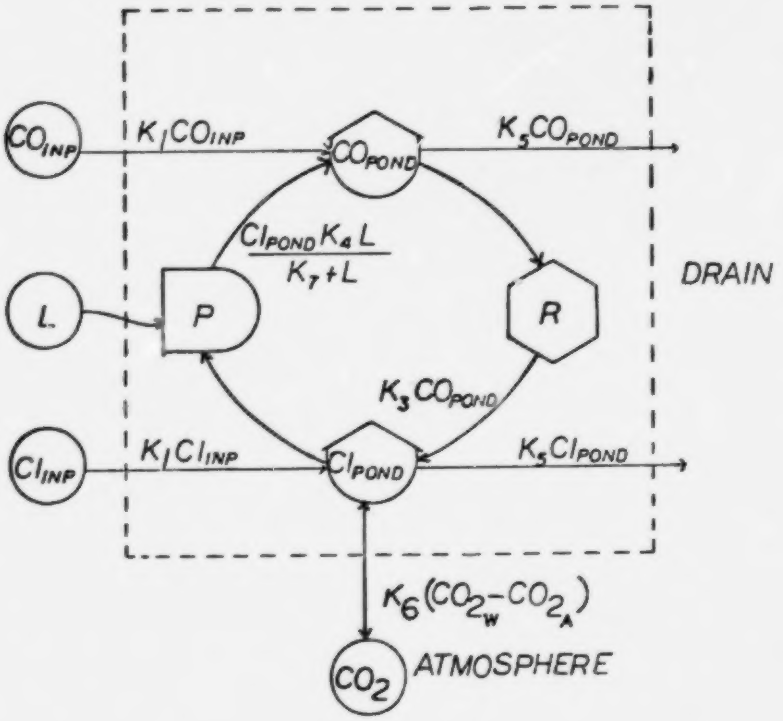
The values of k_1 and k_5 are determined from the rate of water flowing into the ponds. K_3 is the rate of respiration as determined from productivity data. In the expression for photosynthesis L is the light intensity. Light intensity was expressed as a half sine function being zero at night and varying as the sine during the day. K_4 and k_7 are determined from the rate of photosynthesis at high and low light intensities, respectively (Odum, 1970). The diffusion constant, k_6, was calculated as discussed earlier.

To determine the change of organic or inorganic carbon a differential equation is formulated which incorporates those processes which effect the change. The sign for each rate indicates whether it increases or decreases the level of carbon. Two equations for the change of concentration in the pond were formulated.

$$\frac{dCO}{dt} = k_1 CO_{inp} - k_3 CO_{pond} + CI_{pond} \frac{k_4 L}{k_7 + L} - k_5 CO_{pond}$$

$$\frac{dCI}{dt} = k_1 CI_{inp} + k_3 CO_{pond} - CI_{pond} \frac{k_4 L}{k_7 + L} - k_5 CI_{pond} + k_6 (CO_{2a} - CO_{2w})$$

Figure 14. Schematic diagram of pond system in energy circuit language showing reservoirs, rates, and constants. (P, photosynthesis; R, respiration)



The methods selected for the solution of the differential equations are Runge-Kutta and Predictor-Corrector techniques (Conti, 1965). The program is designed to read in initial values for carbon levels and constants. The individual rates are calculated using these values. These rates are then multiplied using matrix algebra to generate an overall change in organic and inorganic carbon. These changes are then fed into the equations for solving the differentials with the subsequent net changes being generated. The change in pH in the ponds is generated from the changes in inorganic carbon.

Figure 15 depicts a set of results obtained from the model. The inorganic and organic carbon varied inversely. The inorganic carbon increases at night and the organic carbon decrease because of respiration. The opposite trend occurred during the light hours due to photosynthesis. The inorganic carbon has a 3.4 mg/l variation. The diurnal variation measured in the ponds ranged from 1 to 4 mg/l. The diurnal variation of organic carbon obtained from tne model was 4.0 mg/l, as compared with measured changes of 2.5 to 5 mg/l. Total carbon diurnal range was 0.5 mg/l from the model while the range of measurements was 2 to 4.6 mg/l.

The results of the model agree fairly well with those obtained from diurnal measurements of carbon levels in the ponds. The variation of total carbon in the ponds was greater than that obtained from the model. The agreement is fairly close when one considers the differing conditions under which the measurements were made and the assumptions used in constructing the model.

The use of CO_{pond} in the rate equation for respiration is justified as long as oxygen levels are not low enough to adversely effect the organisms. This has happened only rarely. In the equation for photosynthesis. $R = CI_{pond} K_4 L/K_7 + L$, the limiting nutrient is usually used rather than the level of inorganic carbon. In the model it is assumed that the rate of change of the limiting factor approximates the change in inorganic carbon, which is normally not limiting. The imput of carbon to the ponds is taken as constant. There is sometimes considerable diurnal variation but over extended periods there is not much variation. Input to the system is important when one is considering extended periods, but daily changes are dominated by respiration and photosynthesis. The light intensity is considered a smooth sine variation and irregularities because of clouds are not taken into

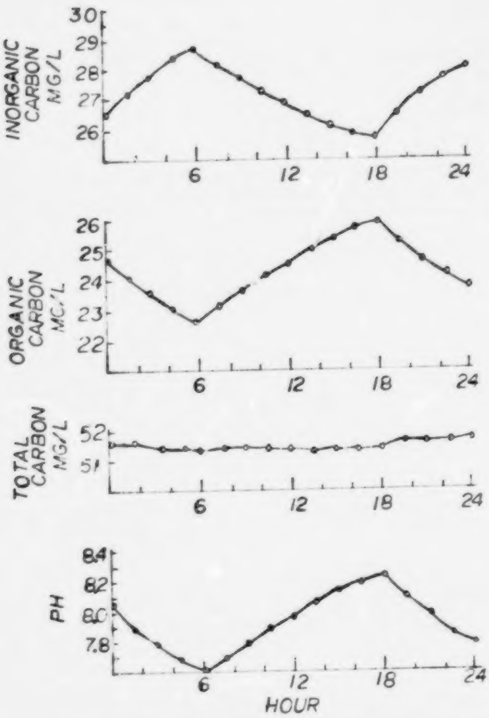


Figure 15. Example of results of model for changes in various carbon species and pH.

consideration. An average value for evaporation was obtained from several months of measurements. For future use, both light and evaporation could be adjusted to approximate the different seasons more closely. The model has proved stable when operated for periods up to 2 weeks.

The diurnal curves from the model take the form of charge-discharge curves commonly encountered in measurements of community metabolism. The rate of charge of either species of carbon is greatest at dawn or dusk. For example, the reservoir of inorganic carbon is increased during the night and is highest just before dawn. When the light function is introduced into the model, the rate of photosynthesis is high because it is dependent on the concentration of inorganic carbon. As this concentration is diminished the rate of photosynthesis decreases and the curve starts to level off. The same type of process takes place at night for inorganic carbon; an initial higher rate of accumulation with a leveling off as the organic pool is used up.

Once a model is deemed a satisfactory representation of the pond system, it can be used in assessing the effects of changes on the system. One of the immediate changes which will be studied is the effect of varying the level of organic carbon. For example, if one were to introduce untreated sewage. Another use of the model will be to determine the effects of sedimentation on carbon flows. An attempt was made to measure the rate of sedimentation in the ponds, but because of physical disturbances in the ponds, it was impossible to obtain any accurate measurements. The model can be used to estimate the contribution of sedimentation to carbon loss.

Simulation using the model can be used to determine the future courses of action of the pond experiment. Such experiments can be used in assessing the effects of treated sewage on estuarine systems. The problem of sewage pollution is mainly one of degree. There should be levels of treated sewage which are compatible with a principle of multiple use of our coastal systems. Questions to be answered are the optimum levels of organics which can be introduced and still maintain the integrity of these estuarine areas and allow a harvest potential for man. Sewage is an energy rich material which, when used properly, could be recycled within the estuarine area, thus enriching it, and allowing greater biomass of organisms useful to man (fishes, crabs, etc.) The level of sustained harvests is another problem which could be investigated. The issue is the extent to which we can enrich coastal areas without detrimental effects.

ACKNOWLEDGEMENTS

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APPENDIX

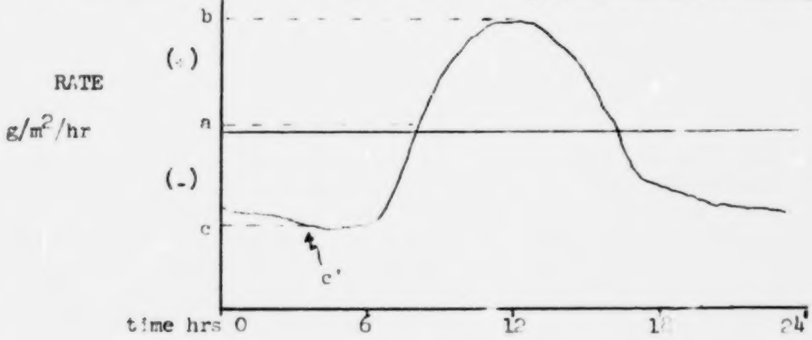
Calculation of Constants for Respiration and Photosynthesis Rate Equations

The constants are calculated using oxygen rate of change curves as shown in Figure 8. Details for constructing such a graph are given in Odum (1956). An idealized curve is shown below.

The photosynthetic rate equation is

$$\text{Rate} = \frac{C_i k_4 L}{k_7 + L}$$

K_4 is significant at high light intensities. If it is assumed that k_7, which is significant only at low light intensities, is small compared to the light intensity at midday, then it vanishes and L in the denominator and numberator cancel. $R = C_i k_4$ is left. The inorganic carbon level, C_i, is known from diurnal measurements and the rate (R) can be read from the graph at b. The equation is then solved for k_4. This value can be inserted in the original equation. The rate at low light intensities can be read from the graph at a and the equation can be solved for k_7. The light intensity can be ensured with a suitable photometer. For the respiration rate equation, $R = C_o$ k, the organic carbon level (C_o) at some point during the night (c' for example) is known from diurnal data and the respiration for that time is read at c. The equation is then solved for k. The units of L and k_7 are cal/m^2/hr and those of $k_4^{'}$ are hr^{-1}. The carbon concentrations are converted to area based by multiplying the concentration (g/m^3) by the average depth of the pond to obtain g/m^2. This gives the correct units for the rate; g/m^2/hr. The units of the respiratory constant are hr^{-1}.



STABILIZATION POND DESIGN CRITERIA FOR TROPICAL ASIA

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INTRODUCTION

Stabilization ponds have been used for wastewater treatment in many countries of the world and are considered most suitable for tropical developing countries. However, Asian governments are only now paying attention to environmental pollution and taking an active interest in feasible wastewater treatment processes. The particular aspects of stabilization pond treatment which make it most applicable to tropical developing countries are the low cost of construction, advantageous climatic factors, ease of operation, and lack of foreign exchange requirements for purchase of mechanical equipment. A major disadvantage of this form of treatment is the large land area requirements associated with ponds. Land use is most efficient if anaerobic and facultative ponds are operated in series. Best use of land area is achieved through maximization of areal BOD removal in any type of stabilization pond. This paper will use areal BOD removal in lb/acre day of 5 day, 20°C BOD, as the most important basis for comparing pond systems. The performance of both anaerobic and facultative ponds as primary units will be considered and the possibilities of series operation of multi-stage anaerobic ponds will be investigated.

ANAEROBIC PONDS

Anaerobic ponds have been found to tolerate higher loadings than facultative ponds and give high BOD removals. However, they can give rise to odours which have often limited their use to remote areas. In Asia, where there has been little or no treatment of sewage or industrial wastes in the past, the aesthetic sensitivity of the population is not so developed as in western countries. Providing that wastes are handled in the future in an organized way at minimum cost and with minimum land use, it is unlikely that Asian people will object to odours. Economic factors are almost certain to override any other consideration in deciding upon a method of wastewater treatment. If the constraint of odourless treatment is eliminated, anaerobic ponds can be designed as highly efficient reactors for BOD removal. Otherwise, operational measures will have to be incorporated into the design and will almost certainly affect the BOD removal efficiency.

In his review of anaerobic pond systems design, OSWALD (1968) discussed the influence of a number of specific environmental factors on the transformations which occur in anaerobic conditions in a pond result from the level of applied BOD loading and that neither nutrient deficiency nor the presence of toxic substances limit the development of a satisfactory microbial population, the anaerobic pond environmental factors most important in the tropics are detention time and pH. The temperature level will normally be conducive to efficient anaerobic breakdown of organic matter, the higher the temperature the more efficient the system performance.

Areal BOD Loading of Anaerobic Ponds

Removal of BOD in anaerobic ponds is brought about both by sedimentation and anaerobic digestion. BOD loading on ponds is often reported as a surface area loading in lb/acre day, this being consistent with the sedimentation function, but a volumetric loading is more appropriate to anaerobic digestion. The BOD loading rate specified should reflect the predominating function but no systematic study of these variables has been reported. Of course, sedimentation will only effect BOD removal if the influent waste contains settleable organic solids. Ponds usually provide sufficient detention time to allow efficient sedimentation. A pond loaded at a rate of 5000 lb BOD/acre day with a weak sewage of 200 mg/l BOD has a surface overflow rate of 69 gal/ft^2 day, which is low for sedimentation tank design. On this basis it seems unreasonable to adopt sedimentation as the overriding consideration in design loading. Since optimum conditions in anaerobic ponds occur in the absence of oxygen and because algae are not involved in the process, oxygen transfer or light reception at the surface cannot be important. The only justification for using areal BOD loading in connection with anaerobic ponds is if biological conversion of organic material suspended or dissolved in the supernatant overlying deposited sludge is dependent on the area of sludge exposed. PARKER *et al.* (1950) and PARKER and SKERRY (1968) pointed out the importance of the sludge layer in BOD removal yet suggested in the earlier paper that a volumetric BOD loading might be the more appropriate parameter. If an anaerobic pond is considered a biological reactor similar to a digester, then volumetric loading is the most useful parameter. Reports on the use of anaerobic ponds for treatment of industrial wastes high in BOD have usually adopted volumetric loading rates and STEFFEN (1968) was most definite in suggesting that loading of this type of pond was a volume characteristic. Specification of loading rate in volumetric terms would allow a trade-off between land cost and construction cost for deep ponds. In spite of volumetric BOD loading and removal parameters being considered the more appropriate forms, the more conventional areal BOD loading and removal rates will generally be used in this discussion. Practical advantages of these forms are that an immediate indication of land use is given and performance results are more directly comparable with those from facultative ponds. It is hoped, however, that future research will be directed towards establishing which is the more rational loading parameter.

Anaerobic pond performance data from various countries as reported in the literature have been collected and are presented in Figure 1 for BOD loadings up to 6000 lb/acre day. Much higher areal BOD loadings have been used in practice with success. For example, BOD loadings up to 46,400 lb/acre day were reported by STANLEY (1966) to give areal BOD removals as high as 37,100 lb/acre

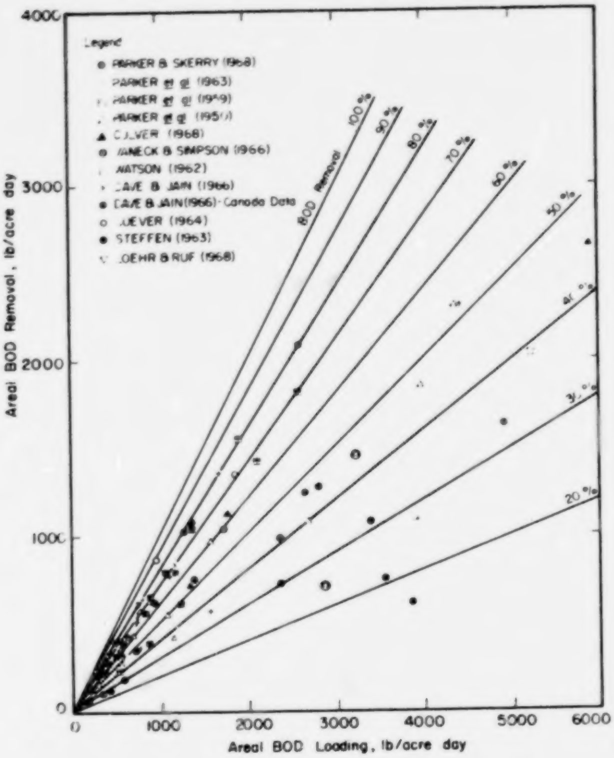

Figure 1. - Reported performance of anaerobic ponds

day in treating packing house waste in deep ponds. The only inference that can be drawn from the plot in Figure 1 is that high BOD loading gives high BOD removal. Further rationalization of these data is not possible because the environmental factors influencing performance are many and their levels have often not been reported. However, many workers have noted a loss of efficiency during winter operation compared with summer, and thus temperature may be considered to have a significant influence on the performance of anaerobic ponds. Because of high ambient temperatures throughout the year, high BOD removals can be expected in ponds in tropical countries, even with high areal BOD loadings. More recent data collected at the Asian Institute of Technology in Bangkok, Thailand, and previously unpublished, are presented in Figure 2 and show the very high areal BOD removals which can be attained with experimental control. Again the trend of high BOD removal at high BOD loading rates is apparent.

The influences of the three factors areal BOD loading, detention time, and influent pH on areal BOD removal were studied by UDDIN (1970) in a factorially designed experiment, each factor at two levels. A plant waste from the production of tapioca starch was fed to 3 ft deep experimental ponds. Table 1 gives the levels of the factors and the BOD removals obtained at steady-state conditions with an average temperature of 77° F. Areal BOD removal was maximum at the highest of the two levels of each of the independent variables. A variance analysis of the data showed the effects of areal BOD loading, influent pH, detention period, and the interaction between loading and influent pH to be significant in the areal removal of BOD (95 percent confidence) in the order given.

Effect of Detention Period on Anaerobic Pond Performance

AMALORPAVAN (1963) suggested that the rate of anaerobic decomposition increases with time, reaches a maximum and then decreases. If this is so, there will be an optimum pond detention period, at any particular BOD loading, above which it will be inefficient to operate. GLOYNA (1965) supported this by stating that detention times longer than five days are not justified because the performance in anaerobic ponds then becomes comparable with that in facultative ponds. He also gave an empirical formulation, as recommended by VINCENT *et al.* (1963), to estimate the degradation action occurring in pretreatment anaerobic ponds in tropical areas, assuming complete mixing and for the pH range 6.8 to 7.2, as follows:

$$P = \frac{P_o}{6\left(\frac{P}{P_o}\right)^{4.8} R + 1} \qquad (1)$$

where, P = Pond and effluent 5-day, 20°C BOD, mg/l
P_o = Influent 5-day, 20°C BOD, mg/l
R = Retention time for completely mixed separate pond system, days

Since this formulation does not include a pond depth term, the predicted performance is not related to areal BOD loading rate but does take account of detention period.

PARKER *et al.* (1950) first presented data demonstrating the influence of detention period on the performance of anaerobic ponds at Werribee, Victoria,

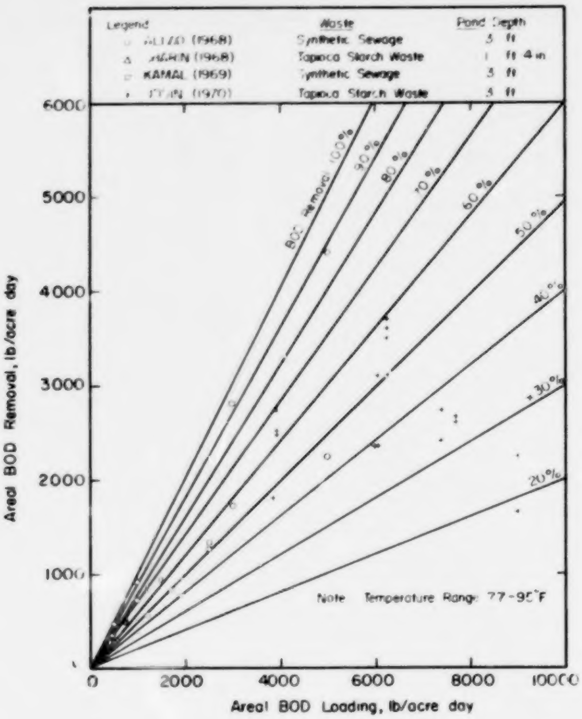

Figure 2. - **Performance of experimental anaerobic ponds in Thailand**

Table 1 – Effect of Loading, Detention and pH on BOD Removal in Anaerobic Ponds

Pond No.	BOD Loading, lb/acre day	Detention Period, days	Influent pH	Influent BOD, mg/1	Effluent BOD, mg/1	BOD Removal lb/acre day	BOD Removal %
1	2100	1	4	260	150	680	32.5
2			7	260	110	1210	57.6
3		2	4	510	310	830	39.3
4			7	510	140	1520	72.6
5	5830	1	4	710	520	1560	26.8
6			7	710	350	2960	50.8
7		2	4	1430	1000	1760	30.1
8			7	1430	600	3380	58.0

Table 2 – Performance of Anaerobic Ponds (after PARKER & others, 1950)

Unit No.	Det. Time days	Area, acre	Period*	BOD Applied lb/day	BOD Loading lb/acre day	BOD Removal* lb/acre day	BOD Removal lb/day	BOD in Effluent lb/day
1	1	0.0198	1	78.7	3980	1862	36.9	41.8
			2	86.9	4390	2309	45.7	41.2
			3	64.8	3270	1469	29.1	35.7
			4	56.8	2870	702	13.9	42.9
			5	53.0	2680	1093	21.7	31.3
2	1½	0.0297	1	41.8	1410	391	11.6	30.2
			2	41.2	1390	326	9.7	31.5
			3	35.7	1200	196	5.8	29.9
			4	42.9	1440	223	6.6	36.3
			5	31.3	1050	196	5.8	25.5
3	2½	0.0495	1	30.2	610	190	9.4	20.8
			2	31.5	637	137	6.8	24.7
			3	29.9	604	167	8.3	21.6
			4	36.3	733	150	7.4	28.9
			5	25.5	515	177	8.8	16.7
4	5	0.0990	1	20.8	210	60	5.9	14.9
			2	24.7	249	169	16.7	8.0
			3	21.6	218	65	6.4	15.2
			4	28.9	292	55	5.4	23.5
			5	16.7	169	93	9.2	7.5
1+2	2½	0.0495	1	78.7	1590	981	48.5	30.2
			2	86.9	1750	1122	55.4	31.5
			3	64.8	1310	706	34.9	29.9
			4	56.8	1150	415	20.5	36.3
			5	53.0	1070	556	27.5	25.5
1+2+3	5	0.0990	1	78.7	795	583	57.9	20.8
			2	86.9	877	628	62.2	24.7
			3	64.8	655	435	43.2	21.6
			4	56.8	574	282	27.9	28.9
			5	53.0	535	365	36.3	16.7
1+2+3+4	10	0.1980	1	78.7	398	324	63.8	14.9
			2	86.9	439	401	78.9	8.0
			3	64.8	327	252	49.6	15.2
			4	56.8	287	170	33.3	23.5
			5	53.0	268	230	45.5	7.5

* from PARKER Paper

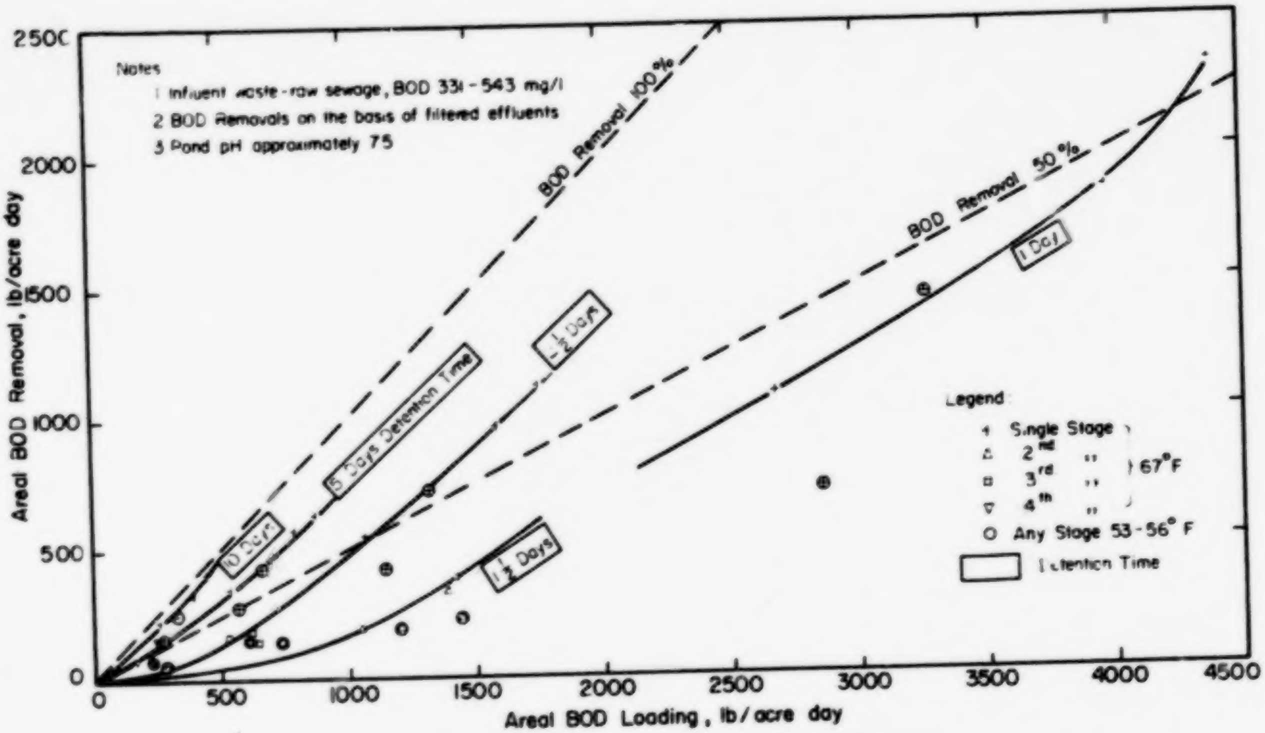

Figure 3. - Effect of detention time on anaerobic pond performance
(Data from Parker et al. 1950)

Australia. Table 2 has been prepared, using their data, to show both areal BOD loading and removal at each stage of anaerobic pond unit and average loadings and removals for multiple stages. Figure 3 presents the results from Table 2 in graphical form and clearly shows the effect of detention period on BOD removal. In view of the fact that high raw waste BOD concentrations are associated with long detention times at any BOD loading, some general conclusions can be drawn from these results. These will apply within the limits of the variables studied and are based on the assumption that pond pH, which was not reported in the original paper, was not an influencing factor. The results show that the effect of a long detention period at any particular loading was to give a higher areal BOD removal than at a short detention period. Also, at any particular detention period a high BOD loading gave higher areal and percentage BOD removals than a low BOD loading. This increasing BOD removal efficiency with increasing loading has important practical implications. Although individual points at low BOD loadings do not fit the curves exactly, BOD removals in 2nd, 3rd and 4th stage ponds seem to be consistent with the trends for 1st stage BOD removals. It appears, therefore, that maintaining high BOD loading rates on all stages of anaerobic pond will give the highest BOD removal efficiency, as well as most economical land use.

Results collected by ALIVIO (1968) in anaerobic pond studies in Bangkok, using synthetic sewage as influent waste to 3 ft deep experimental ponds, are presented in Figure 4. Although gross interpolation was necessary in the figure, and there is no justification for using individual points for practical design, the trends suggested by the few data collected can be of general use. As with Parker's results, Figure 4 indicates that increased detention time at any BOD loading gives increased BOD removal but the effect is more dramatic at high BOD loading levels. Likewise the results suggest that at any detention period the highest BOD removal would result from the highest BOD loading. It is also interesting to note the close agreement of points for the 200 mg/l influent BOD in this figure with Parker data presented in Figure 3.

UDDIN (1970) carried out a study of anaerobic pond treatment of tapioca starch waste in 3 ft deep experimental ponds and obtained the results shown in Figure 5 at steady-state conditions. Four replicated ponds receiving waste with average BOD 3800 mg/l and loaded at 6160 lb/acre day with 5 days detention period gave a mean areal BOD removal rate of 3570 lb/acre day (standard deviation 70 lb/acre day). The two lines have been drawn on Figure 5 to represent approximate BOD removals for areal BOD loadings of 6000 and 4000 lb/acre day. It will be noted that a BOD loading near 6000 lb/acre day gave maximum BOD removal at nearly all detention times. This can be explained when reference is made to Figure 6 showing the respective pH levels in the ponds at the various loading rates and detention times. At BOD loadings greater than 6000 lb/acre day, with this particular waste, effluent pH dropped below 6.0 in most ponds which certainly affected performance. The exception is a loading of 7700 lb/acre day at 2 days detention time which maintained a pH of 6.3 and gave a better BOD removal than the loading of 6020 lb/acre day at the same detention time. In spite of the fact that pH was an influencing variable in this study, the trend of the data confirms that increasing detention time, up to 5 days for

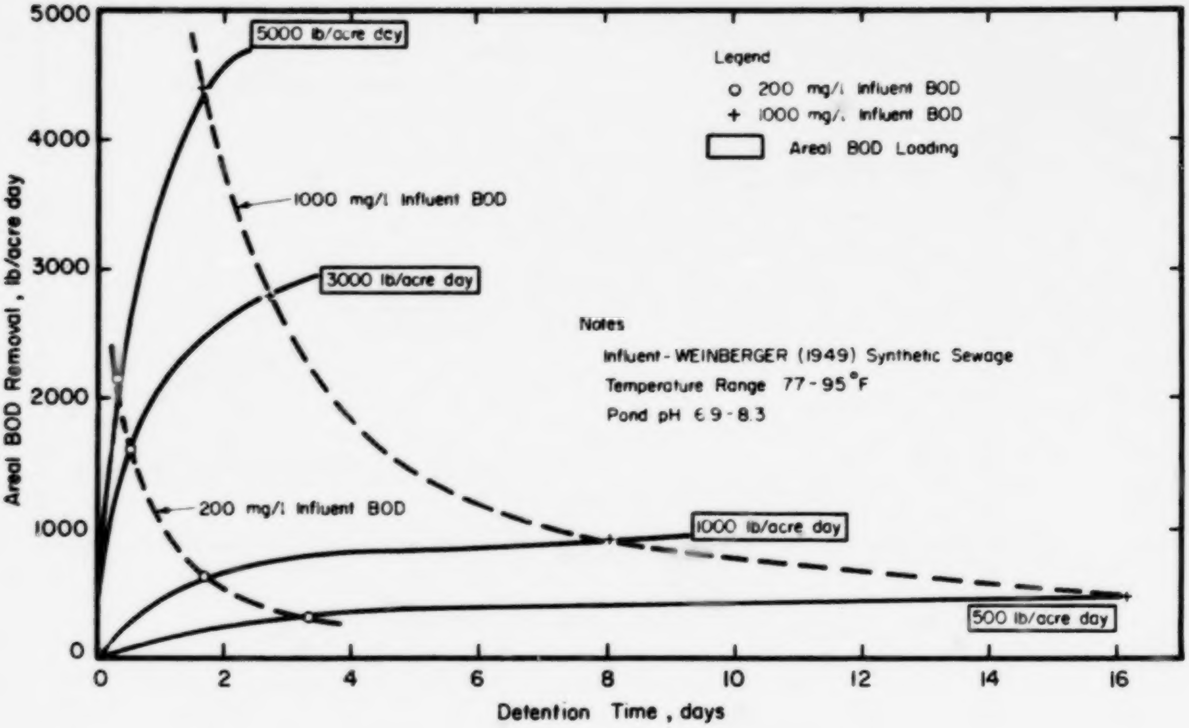

Figure 4. - Effect of detention time on BOD removal in anaerobic ponds (after alivio, 1968)

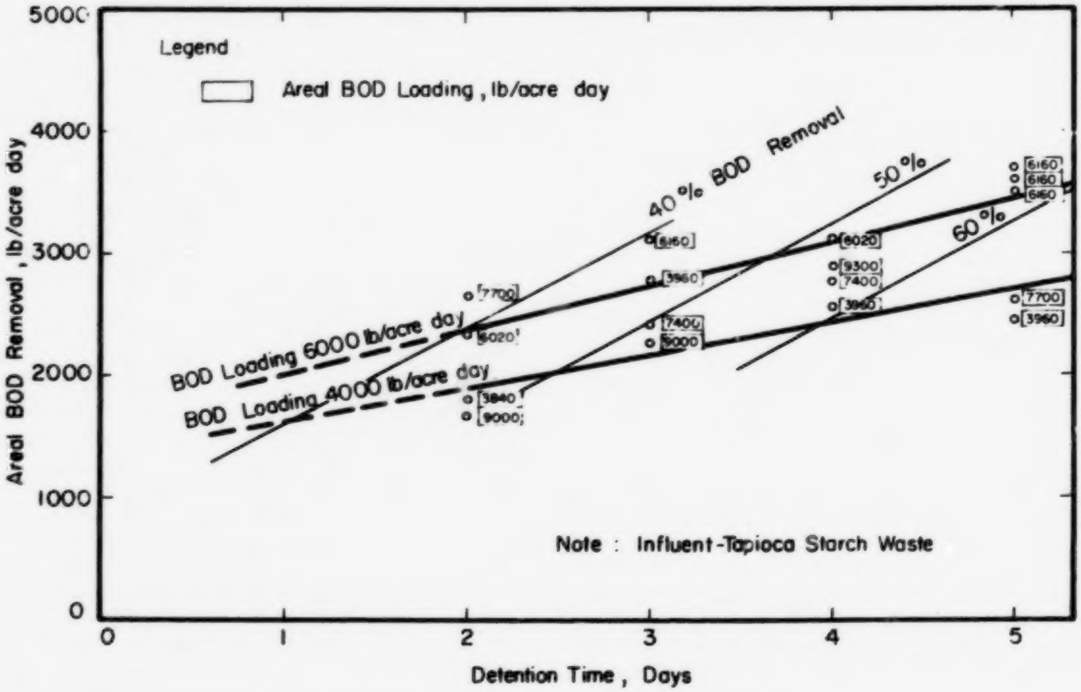

Figure 5. - Effect of detention time on single-stage anaerobic pond performance

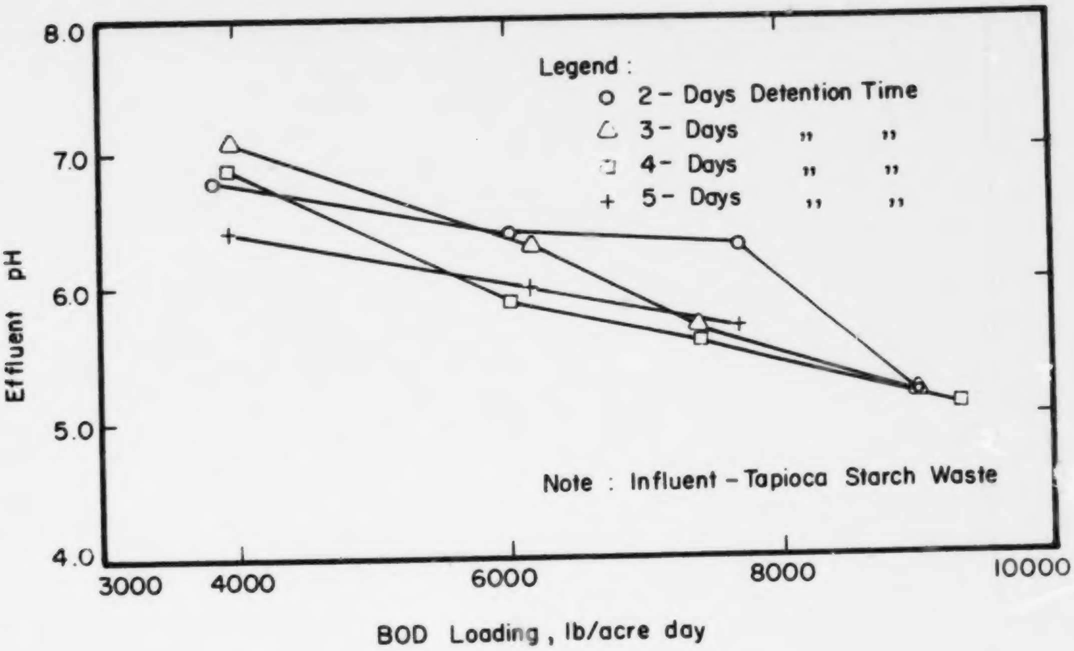

Figure 6. - Effect of BOD loading on anaerobic pond effluent pH.

this waste, at a particular BOD loading will result in increased BOD removal. Also, until pH became the limiting factor, apparently at levels below 6.0, increasing BOD loading gave improved areal BOD removal at any detention time between 2 and 5 days. If Uddin's results are compared with those of Parker and Alivio it will be seen that the trends are similar with the exception that the effect of pH on pond performance is significant. It would appear that the higher temperature in the tropics allows anaerobic fermentation to proceed at a rapid rate at pH levels down to 6.0, which is lower than the minimum practical limit suggested by OSWALD (1968).

The results reported are too few on which to base a formulation of the effect of detention period on anaerobic pond performance at the present time, but do enable general conclusions to be drawn. Increased detention time at any particular BOD loading results in increased BOD removal efficiency, the magnitude of the increase being greatest at high loadings. The highest possible areal BOD loading at any particular detention period will give the greatest areal BOD removal efficiency, providing pond pH does not become limiting. Strictly speaking, it can only be stated that this will apply within the range of detention times covered in the tests. In practical terms, for anaerobic pond treatment of a waste with a particular BOD concentration, applying the raw waste at the maximum areal BOD loading which will permit pond pH to remain above 6.0 in the tropics will give the highest areal BOD removal efficiency, always assuming odour is not a consideration. Waste dilution through recirculation or other operational technique will reduce detention time and hence can be expected to adversely affect areal BOD removal.

Series Operation of Multi-Stage Anaerobic Ponds

Three stages of 3 ft deep experimental anaerobic ponds were operated by UDDIN (1970) in the treatment of tapioca starch waste, with an average BOD concentration of 3800 mg/1, under four loading systems, the equivalents of which are shown diagrammatically in Figure 7. A summary of his performance results is given in Table 3, which includes average loading and removal rates for multiple stages of ponds in series. Comparing system I with systems II, III and IV in Table 3, it can be seen that the effect of the higher 2nd stage areal BOD loading was to give a much higher areal BOD removal and a slightly higher removal efficiency, in spite of the lower detention time. A comparison of 2nd stage areal BOD loading was to give a much higher areal BOD removal and a slightly higher removal efficiency, in spite of the lower detention time. A comparison of 2nd stage areal BOD removals with first-stage areal BOD removals for systems II, III and IV show that even at a slightly higher areal BOD loading, the removal was lower. However, it seems the lower areal BOD removal was due not to the fact that the organic matter became less amenable to breakdown (decreasing K value) after passage through the first stage pond, but to the lower detention time in the 2nd stage ponds. The BOD removal of 2560 lb/acre day in the 2nd stage ponds in systems II, III and IV is consistent with that to be expected from Figure 5 at a detention period of 2 days. In system I, 2nd stage BOD removal was lower than would be predicted from Figure 5

Table 3 – BOD Removals in Series Anaerobic Ponds

System	Pond Stage	Area, acre	Detention Time, days	Input		Output		BOD Removal	
				BOD lb/day	BOD Loading lb/acre day	Effluent BOD, mg/l	BOD lb/day	Areal, lb/acre day	Efficiency, %
I	1	1.47	5.0	9060	6160	1630	3860	3510	57.0
	2	1.00	3.4	3860	3860	1000	2430	1430	37.0
	3	1.00	3.4	2430	2430	530	1300	1130	47.0
	1+2	2.47	8.4	9060	3670	1000	2430	2690	73.2
	1+2+3	3.47	11.8	9060	2610	530	1300	2240	85.6
II	1	2.35	5.0	15580	6160	1520	6570	3680	57.8
	2	1.00	2.0	6570	6570	990	4010	2560	39.0
	3	3.34	4.7	4010	1200	630	2540	440	36.4
	1+2	3.53	7.0	15580	4410	990	4010	3270	74.2
	1+2+3	6.87	11.7	15580	2270	630	2540	1900	83.8
III	1	2.53	5.0	15580	6160	1520	6570	3680	57.8
	2	1.00	2.0	6570	6570	990	4010	2560	39.0
	3	4.18	5.5	4010	960	570	2300	410	42.5
	1+2	3.53	7.0	15580	4410	990	4010	3270	74.2
	1+2+3	7.71	12.5	15580	2020	570	2300	1720	85.3
IV	1	2.53	5.0	15580	6160	1520	6570	3680	57.8
	2	1.00	2.0	6570	6570	990	4010	2560	39.0
	3	6.90	9.3	4010	580	410	1650	340	58.6
	1+2	3.53	7.0	15580	4410	990	4010	3270	74.2
	1+2+3	10.43	16.3	15580	1490	410	1650	1335	89.3

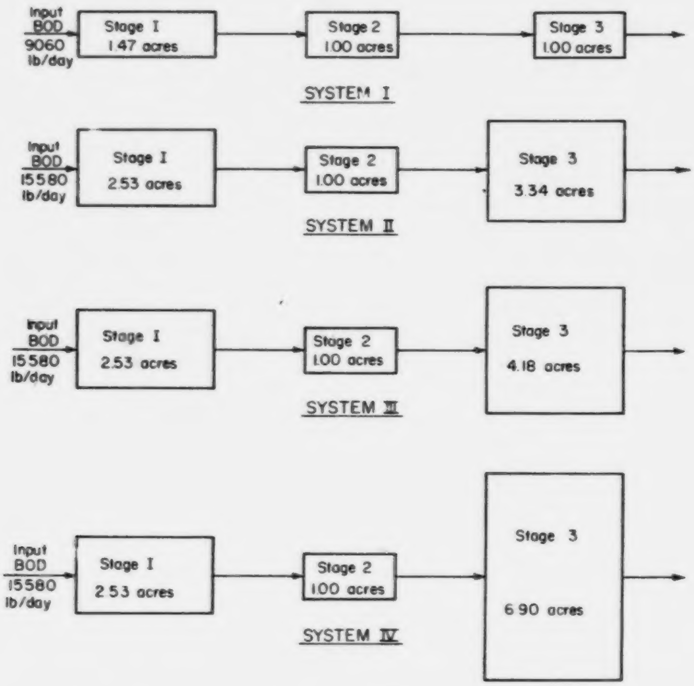
Input BOD 9060 lb/day
Stage I 1.47 acres
Stage 2 1.00 acres
Stage 3 1.00 acres
SYSTEM I
Input BOD 15580 lb/day
Stage I 2.53 acres
Stage 2 1.00 acres
Stage 3 3.34 acres
SYSTEM II
Input BOD 15580 lb/day
Stage I 2.53 acres
Stage 2 1.00 acres
Stage 3 4.18 acres
SYSTEM III
Input BOD 15580 lb/day
Stage I 2.53 acres
Stage 2 1.00 acres
Stage 3 6.90 acres
SYSTEM IV

FIGURE 7 - EQUIVALENT SYSTEMS OF ANAEROBIC PONDS IN SERIES

Figure 7. - Equivalent systems of anaerobic ponds in series

for a detention period of 3.4 days, this implying either that the organic matter was becoming more difficult to degrade or that the difference was due to sedimentation of organic particles in the first stage pond. Further studies are being carried out to establish the true effect and cause. 3rd stage areal BOD removals are generally low due to low applied BOD loadings but an increasing efficiency of BOD removal is noted with increase in detention time, in spite of decreasing areal BOD loading. In systems III and IV, effluents from the 3rd stage ponds showed dissolved oxygen concentrations of 0.5 and 1.5 mg/1, respectively, at mid-day indicating that these ponds were operating at the lower loading limit of what might be termed anaerobic ponds or at the upper loading limit of facultative ponds since algae were present in the surface layer.

Plots of average areal BOD loadings, average areal BOD removals, and effluent BOD concentrations for the four systems of series ponds are shown in Figure 8. For the first two stages of ponds, average areal BOD removals were highest in systems II, III and IV, where average areal BOD loading rates were highest, although total detention times were less. However, system I gave the highest average areal BOD removal after three stages of ponds because of the higher average loading rate, even at a similar total detention time to those in systems II and III. System IV with the lowest average areal BOD loading gave the lowest average areal BOD removal although it produced a slightly better effluent. The average BOD removals in these systems may be compared with Alivio's results in Figure 4, where it will be seen that the removals are lower for this waste at any average areal BOD loading than for synthetic sewage at the same loading and detention time. This is probably a result of the lower pH in the case of the systems treating tapioca starch waste. Allowing for the effect of pH, the results suggest that the same order of removal can be obtained with lower BOD wastes and that the general trends would equally apply.

Recent results from series operation of multiple stage anaerobic ponds confirm the suggestion, previously mentioned in connection with Parker's data, that maintaining high BOD loading rates on all stages of ponds will give best areal BOD removal, and hence most efficient land use. If a constant high areal BOD loading is applied to each stage of anaerobic pond, then detention time in each stage will decrease from first to last stage and pond area will likewise decrease. This is a new practical concept which might well receive attention in future anaerobic pond systems design. Furthermore, it is postulated that stages after the first might well be loaded at areal BOD loadings of increasing magnitude, to maintain a high average areal BOD loading, because pH increases in passage through any stage allowing greater loadings to be applied. These suggestions are based on limited data and their applicability will be dependent on the ponds being operated at an average areal BOD loading within the capability of an anarobic pond system, the upper loading limit not being known at present, and at minimum detention times in later stages which do not limit the efficiency of BOD removal.

To illustrate the possibilities for treating 1 MGD of tapioca starch waste, with mean influent BOD concentration 3800 mg/1, in anaerobic ponds, the performance of the alternative prototype systems shown in Figure 9 have been estimated using Figure 5. Pond BOD loading was taken as 6000 lb/acre day until the required detention time in a stage became less than 0.86 days, necessitating undesirable extrapolation for estimation of BOD removal, when a loading of 4000 lb/acre day was adopted. System B with stepped addition of waste was introduced in an attempt to maintain minimum detention periods within the limits of the results in Figure 5 for as many pond stages as possible. It was assumed that the effluents from the anaerobic pond systems would be treatable in a facultative pond if a high quality effluent was required. The results are presented in Table 4, in which it can be seen that system A, with the total input flow applied to the first and subsequent ponds, was most efficient in removing BOD. The least area was required to give an average BOD removal of 2800 lb/acre day at an average BOD loading of 3000 lb/acre day. Stepped addition of waste in system B, in spite of allowing a BOD loading of 6000 lb/acre day to be adopted for 5 stages, gave less average BOD removal than system A. There is no doubt that a flow diagram and loadings similar to those adopted in system A would approach the best practical anaerobic pond treatment system for this waste. It is highly probable that a similar flow diagram with different loadings, appropriate to the particular waste being treated, would be generally applicable for anaerobic pond treatment of any waste. Further research is under way to fully justify this suggestion.

Odour from Anaerobic Ponds

Heavily loaded anaerobic ponds will undoubtedly produce odours, particularly if environmental conditions are suitable for the liberation of hydrogen sulfide. The cause of the odour emitted from anaerobic ponds is the release of hydrogen sulfide, ammonia, lower hydrocarbons and other organic compounds in gaseous form. Hydrogen sulfide equilibrium, which is pH dependent, is a critical factor in the production of odour in anaerobic systems. The effect of odours may be to limit the use of anaerobic ponds to remote areas, but this is less likely in developing countries. In tropical developing countries at the present time, very strong industrial wastes, such as those released from the processing of agricultural materials, will normally be producing odours because of irresponsible discharge without treatment. Organized and efficient treatment before discharge will be an improvement of general environmental conditions even if odours are produced in the process. The efficiency of the anaerobic form of pond in removing BOD is likely to be the main factor in deciding upon the type of treatment adopted by industry. Difficulty of land acquisition and efficiency of land use will often force municipal authorities to adopt anaerobic ponds for treatment of sewage in the tropics, recognizing that facultative ponds will be used for final polishing of the effluent. Therefore, if the assumption is made that anaerobic ponds will be widely used in tropical countries, the possibilities available for control of odours must be considered.

Control of odour can be accomplished by reducing pond loading to a point where dissolved oxygen begins to be present at the surface, through algal photosynthesis. Although the BOD loading level at which this occurs in the tropics is higher than in a temperate climate, its adoption will still limit the extent of land use efficiency. In Bangkok, UDDIN (1970) detected dissolved oxygen at the pond

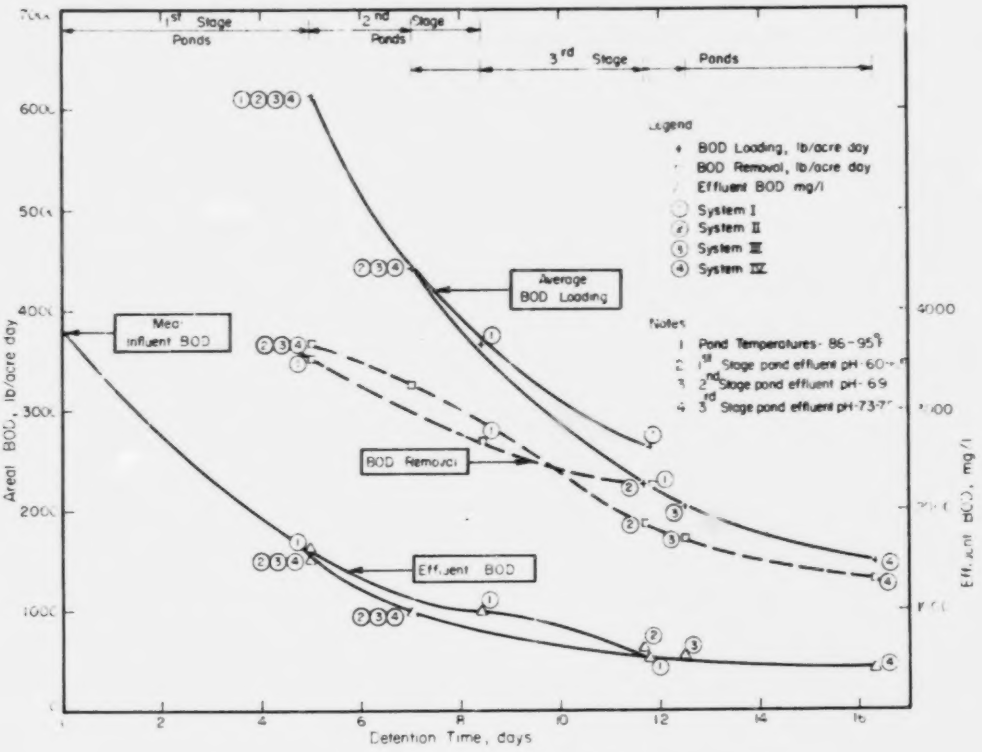

Figure 8. - Series operation of anaerobic ponds treating tapioca starch waste

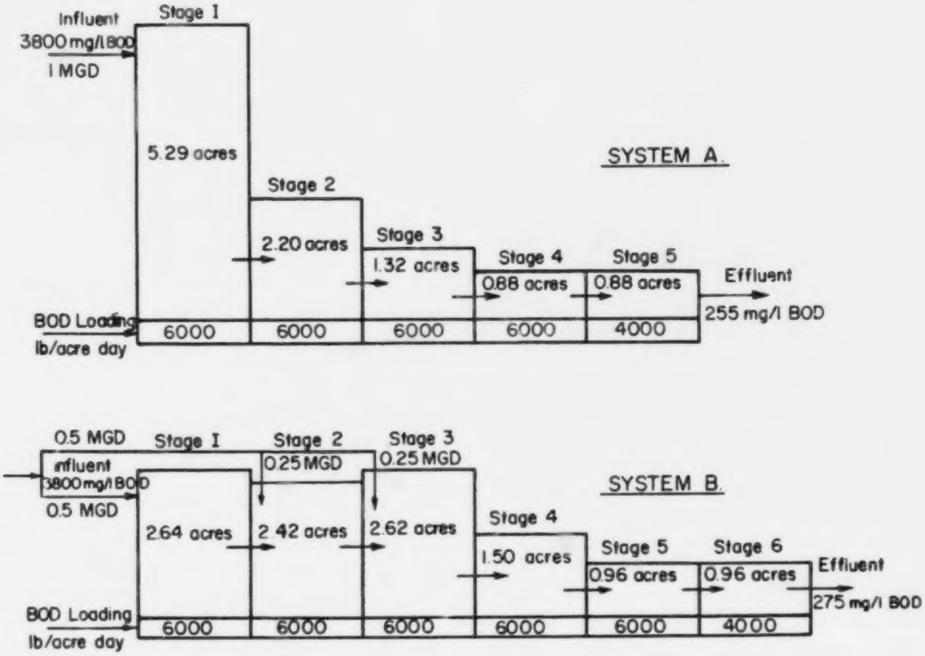
SYSTEM A.
Influent
3800 mg/l BOD
1 MGD
Stage I
5.29 acres
Stage 2
2.20 acres
Stage 3
1.32 acres
Stage 4
0.88 acres
Stage 5
0.88 acres
Effluent
255 mg/l BOD
BOD Loading lb/acre day
6000
6000
6000
6000
4000
SYSTEM B.
0.5 MGD
Influent
3800 mg/l BOD
0.5 MGD
Stage I
2.64 acres
Stage 2
0.25 MGD
2.42 acres
Stage 3
0.25 MGD
2.62 acres
Stage 4
1.50 acres
Stage 5
0.96 acres
Stage 6
0.96 acres
Effluent
275 mg/l BOD
BOD Loading lb/acre day
6000
6000
6000
6000
6000
4000

Figure 9. - Alternative anaerobic pond systems for tapioca starch waste treatment

Table 4 – **Alternative Prototype Anaerobic Pond Systems for Tapioca Waste Treatment**

Alternative System	Pond Stage	Input		Required Area, acres	Detention Time, days	BOD* Removal lb/acre day	Output	
		BOD, lb/day	BOD Loading lb/acre day				BOD lb/day	Effluent BOD mg/1 +
A	1	31,710	6000	5.29	5.18	3500	13,200	1580
	2	13,200	6000	2.20	2.15	2400	7,920	950
	3	7,920	6000	1.32	1.29	2100	5,220	625
	4	5,220	6000	0.88	0.86	1900	3,540	425
	5	3,540	4000	0.88	0.86	1600	2,130	255
	1+2+3+4+5	31,710	3000	10.57	10.33	2800	2,130	255
B	1	15,855	6000	2.64	5.18	3500	6,600	1580
	2	14,530	6000	2.42	3.16	2800	7,750	1240
	3	15,680	6000	2.62	2.56	2550	8,980	1080
	4	8,980	6000	1.50	1.47	2150	5,760	690
	5	5,760	6000	0.96	0.94	2000	3,820	460
	6	3,820	4000	0.96	0.93	1600	2,280	275
	1+2+3+4+5+6	31,710	2860	11.10	14.24	2650	2,280	275

*From Figure 5, for 3 ft deep ponds
+Assuming no evaporation or percolation.

surface only at BOD loadings of 960 and 580 lb/acre day, whereas a loading of 1200 lb/acre day gave completely anaerobic conditions. If odour was not considered, a BOD loading of 6000 lb/acre day could be applied. An areal BOD loading of about 1000 lb/acre day seems to provide a slight aerobic surface layer in the tropics and would be the limit of completely anaerobic operation.

Another important factor connected with pond loading is pH. Heavily loaded ponds will tend to have low pH and this is one environmental condition which allows release of hydrogen sulfide if the pond contains sulfur. The hydrogen sulfide-sulfide equilibrium is such that if pond pH is maintained above 8.0 the partial pressure of the small amount of free hydrogen sulfide present will be insignificant and odour problems will not occur. pH adjustment in combination with BOD loading limitation would be an effective means of preventing odours but would increase either land use or operational costs, or both.

An alternative form of odour control which has been used in anaerobic ponds is chemical treatment, sometimes in combination with surface aeration. CANHAM (1968) reported on the use of sodium nitrate in the canning industry, but the costs involved would normally be prohibitive for widespread use in developing countries.

The other measure which has been adopted to overcome odour problems in anaerobic ponds is recirculation. ABBOTT (1962) reported odourless operation at a BOD loading rate of approximely 5800 lb/acre day on the anaerobic pond of the anaerobic-aerobic pond system at Wynberg-Muizenberg, South Africa, when secondary aerobic pond effluent was recycled into the incoming sewage in the ratio 1 to 1. VAN ECK and SIMPSON (1966) reported primary anaerobic pond BOD loadings from 1692 to 5214 lb/acre day on the Chatsworth, Durban, South Africa ponds to give odourless operation in summer and winter when 38.8 to 13.9 percent of the effluent from the aerobic pond was recycled to the incoming sewage. KAMAL (1969) compared anaerobic pond system performance with and without recirculation at BOD loadings of 1500 and 2500 lb/acre day, not including recycled BOD, using synthetic sewage as influent with a BOD concentration of 200 mg/1. The highest BOD removal rate recirculation, for recirculation ratios between 5 and 100% of inflow. It seems likely, therefore that recirculation will be effective in suppressing odours in anaerobic pond operation if a facultative pond effluent containing dissolved oxygen is recycled but inevitably pond areal BOD removal will decrease. An exception to this might be if the recycled flow had a significantly higher pH than the influent flow to an anaerobic pond, because the influence of pH on areal BOD removal is greater than that of detention time. It should be relatively easy in tropical countries to maintain the recycled flow at the pond surface, particularly with deep ponds, because the effluent will normally be at a higher temperature than the influent.

FACULTATIVE PONDS

As the major purpose of facultative stabilization pond treatment is removal of organic material with minimum odour nuisance, the following discussion is not related to pathogen survival but primarily describes the development of design criteria for primary facultative ponds in tropical climates on the basis of BOD removal. In contrast with other forms of sewage treatment, the major proportion of the costs of treatment by ponds is associated with land requirements. It is natural for design parameters to reflect these requirements in preference to relating to volumetric measures, as in the case of conventional secondary treatment plants. It is for this reason that the areal BOD loading rate has come into widespread use as a design parameter. On the other hand BOD removals have largely been reported as percentages which has made the comparison of pond efficiencies difficult. The areal unit, lb BOD removal/acre day, used in conjunction with areal BOD loading reflects not only the efficiency of BOD removal but also of land usage.

Important external factors influencing the primary facultative pond process are areal organic loading, pond depth, detention period, temperature, solar radiation intensity, and the quantity and quality of influent waste. Although consideration has been given to the influences of detention period and temperature by MARAIS and SHAW (1961) and GLOYNA (1965) which resulted in suggested design formulations, little theory has been developed for other influencing factors. In comparison with other secondary sewage treatment processes the facultative oxidation pond is far more complex than its rather simple physical exterior suggests. Both anaerobic and aerobic phases within the pond are extremely complex involving both plants and a wide range of animal life. Several workers have studied pond systems in a rational way such as SUWANNAKARN (1963), SHAW *et al.* (1962) and MARAIS and SHAW (1961). Few researchers have concentrated on principles involving the fundamental interactions and relationships between biological phases within the facultative pond, as has OSWALD (1957).

A review of the literature pertaining to both tropical and temperate facultative oxidation pond research and experience has been carried out. Although the separate influences of the more usual design criteria (areal BOD loading, pond depth, detention period and temperature) are difficult to define clearly, each is discussed below.

Areal BOD Loading of Facultative Ponds

Data collected from primary facultative ponds operating under 143 different conditions have been collated and analyzed. In general, BOD removals in these ponds remained within the range of 70 to 90% (mean 79.4%, standard deviation 7.2%). Notable exceptions to this generalization were ponds located near Lebanon, Ohio operated at areal organic loadings in excess of 100 lb BOD/acre day (HORNING *et al.*, 1964). The influence of areal loading on areal BOD removal my be visualized using Figure 10*. Little scatter of data occurs about the regression line:

$$L_r = 9.23 + 0.725\, L_o \qquad (2)$$

(Standard Error of Estimate 14.9
lb BOD/acre day)

where, L_r = Areal BOD removal, lb/acre day, and
L_o = Influent Waste BOD, mg/l

This regression equation has a very high correlation coefficient of 0.995 and a 95% confidence interval of ± 29.3 lb BOD/acre day removal. It was expected that the regression line would pass though the intersection of the axes, which prompted a second regression analysis to be performed with the line passing through the origin. The resultant error estimate was larger than before, the correlation coefficient being 0.95, Equation 2 is preferred for estimation of expected BOD removals at any loading between 30 and 500 lb BOD/acre day.

A rather startling suggestion which this analysis reveals is that BOD removal in facultative ponds may be independent of pond depth, detention period, and influent waste concentration. The effect of temperature also appears to be minimized, but it is to be noted that temperatures were not evenly distributed over the range of loadings, higher ambient temperatures being associated with higher loadings.

Effect of Detention Period on Facultative Pond Performance

The effect of detention period on areal BOD removal is shown in Figure 11; but, unfortunately, this effect is confounded with that of areal BOD loading. Indeed, the sudden rise in removal rate at low detention times is quite contrary to expectations and is possibly due to the effect of high BOD loadings at reduced detention periods. As organic loading has been shown to have considerable effect on areal removal it is postulated that detention period has little significant influence on removal. These data were analyzed in terms of percentage BOD removals in an attempt to reveal the separate effect of detention period. The influence of varying temperature was avoided through partitioning the data into ambient temperature ranges. Where sufficient temperature data were not available, the corresponding pond observations were omitted from the analysis. Under the assumption that confoundment of temperature and detention period effects were removed by data partitioning, Figure 12 should give a reasonably clear and independent evaluation of detention period effects. Regression equations and relevant statistics are listed in Table 5. Detention period appears to affect percentage BOD removals, however in view of the wide scatter of data about other regression lines this could only be proven within the 50 to 60°F temperature range.

O'CONNOR and ECKENFELDER (1960) have utilized the monomolecular equation as a basis for aerated lagoon operation as have MARAIS and SHAW (1961) for primary facultative ponds in central and southern Africa. Based on the assumption that ponds are completely mixed by natural (MARAIS, 1966) or artificial (O'CONNOR, 1960) means a materials balance may be performed as follows:

$$L_a Q - L_e Q = (K L_e) V \qquad (3)$$

where, L_e = Concentration of effluent BOD, mg/l
L_a = Concentration of influent BOD, mg/l
Q = Rate of influent flow, ft^3/day
V = Pond volume, ft^3
K = Reaction rate constant to base 10, day^{-1}

which may be expressed as:

$$L_e/L_a = 1/(1+KD) \qquad (4)$$
$$\text{or } L_a/L_e - 1 = KD$$

MARAIS and SHAW (1961) adopted Equation 4 for primary facultative oxidation pond design in central and southern Africa and gave a tentative value of K = 0.17.

To study this theory over a wide range of pond conditions, values of $L_a/L_e - 1$ were regressed against detention period for each range of temperature. The slopes of least squares regression lines passing through the origin were best approximations of K values in each case. Plots of data revealed wide scatter as is indicated by the confidence intervals listed in Table 6.

If detention period strongly influenced BOD removal in

Table 5 – Regression Statistics of Relationships between BOD Removals and Pond Detention Period

Temperature Range °F	Regression Equation	Correlation Coefficient	Std. Error of Estimate	No. of Data Points
0-30	$P=70.1+0.30\ D^{*+}$	0.443^{**}	5.45	9
30-40	$P=76.8+0.24\ D^{+}$	0.421^{**}	4.95	8
40-50	$P=64.3+0.41\ D^{+}$	0.436^{**}	12.19	9
50-60	$P=71.3+0.28\ D$	0.469	5.48	17
60-70	$P=77.4+0.20\ D^{+}$	0.324	6.36	34
70-80	$P=79.3+0.11\ D^{+}$	0.326	6.05	22
80-100	$P=75.1+1.30\ D^{+}$	0.651	6.90	22

* where, P = Removal of BOD through pond, %
D = Detention period, days
** Correlation not proven significant (95% confidence)
+ Effect of detention period on P not proven significant (95% confidence)

Table 6 – Evaluation of Reaction Rate Constants for Facultative Ponds

Temp. range °F	Rate Constant K, day^{-1}	Std. Err. of Est.	95% Confidence Interval	Number of Data Points
0-30	0.146	1.40	± 3.2	9
30-40	0.190	1.86	± 4.4	8
40-50	0.135	2.60	± 6.0	9
50-60	0.144	1.60	± 3.4	17
60-70	0.225	6.86	±13.4	34
70-80	0.162	5.27	±11.0	22
80-100	1.604	5.89	±12.2	22

facultative ponds, as Equation 4 suggests, only a slight scatter about regression lines would be expected. Such was not the case.

Based on the van't Hoff-Arrhenius relationship:

$$t/t_o = \theta^{(T_o - T)} \qquad (5)$$

where, t = Required reaction time, days at temperature T, °C
t_o = Base reaction time, days at temperature T_o, °C, and
θ = Temperature coefficient

GLOYNA (1965) has developed a pond design formulation utilizing a pond detention period of 3.5 days at 35°C as a base optimum achieving 90% BOD reduction. Hence, using D to represent detention period:

$$D = 3.5\,\theta^{(35-T)} \qquad (6)$$

Assuming that the required detention period is proportional to influent waste BOD concentration, a straight line factor $L_a/200$ was introduced into Equation 6, where L_a is influent BOD and 200 mg/l was considered to be the average BOD value of sewage in the United States, giving:

$$D = 3.5\,\theta^{(35-T)}\,L_a/200 \qquad (7)$$

using, $D = V/N_pQ$
where, V = Volume of pond, acre ft
Q = Sewage flow, US gal/day
N_p = Number of people served

then, $$V = CN_p\,Q\,L_a\,\theta^{(35-T)} \qquad (8)$$

where, $C = 5.37 \times 10^{-8}$, a constant adjusting units and incorporating the ratio 3.5/200

Equation 7, being the source of Equation 8, was used to calculate theoretical detention periods for operating ponds which have been reported to achieve 85 to 95% BOD removals. The standard deviation of differences between actual and theoretical detention periods, as calculated using Equation 7, was 16.1 days and 95% confidence limits ± 31.5 days.

The monomolecular and van't Hoff–Arrhenius relationships are recognized to have possible application to many sewage treatment processes. However, the complexity of oxidation pond symbioses suggests that formulations based only on these principles utilizing detention period, temperature and rate constants as the most influential operating parameters are not likely to represent pond processes over a wide range of operating conditions. A resort to simplified relationships derived from basic principles may have some value when applied to a limited range of conditions but there is some danger that unjustified extrapolations of these relationships will be used by design engineers in the field.

Importance of Depth in Facultative Pond Operation

Reports on studies of the effects of depth are scarce. The effects of depth variation are likely to be confounded

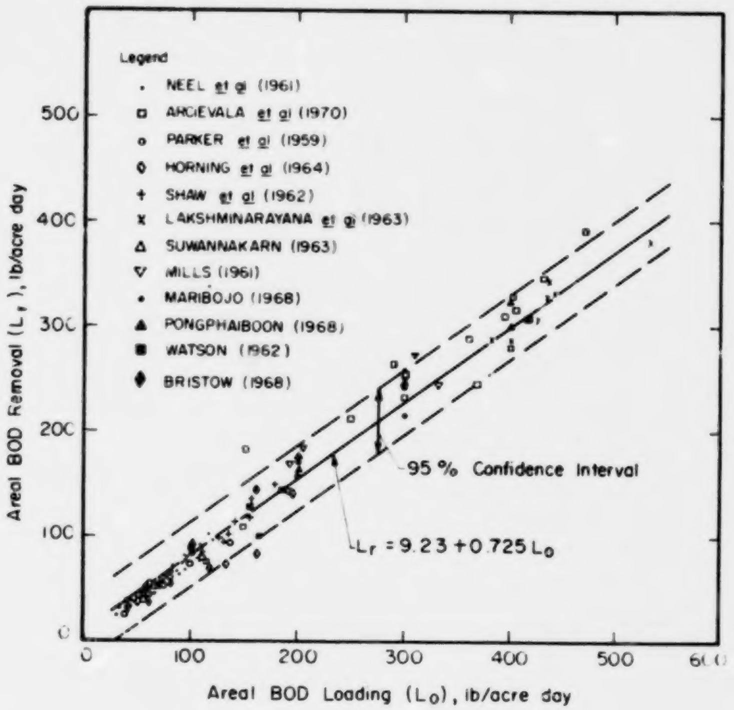

Figure 10. - Areal BOD removal as a function of BOD loading

*Unpublished Indian CPHERI data provided through kind permission of A. C. Arcievalla, CPHERI.

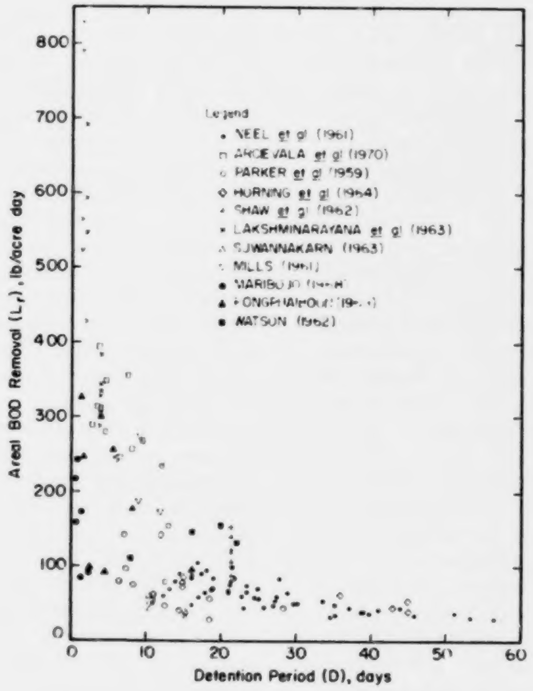

Figure 11. - Pond detention periods and areal BOD removals

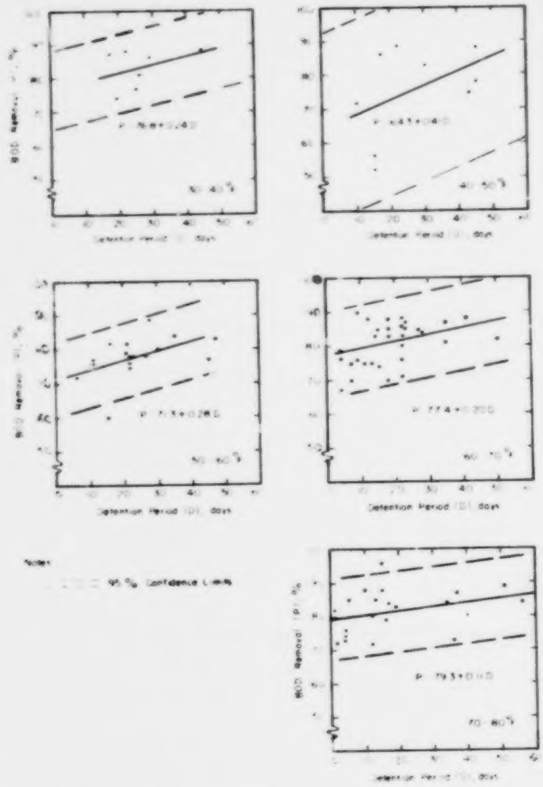

Figure 12. - The influence of detention period over % BOD removal in facultative ponds

with those of detention period. Under constant influent flow conditions a pond's depth change will effect a proportional change in detention period. MILLS (1961), using experimental ponds at 2.5 and 3.5 feet, found that pond depth had no effect on pond operating efficiency. Likewise PONGPHAIBOON (1968) observed no significant effect of depth variation from 0.83 to 3 ft on BOD removal efficiencies at areal BOD loadings of 100, 200 and 300 lb/acre day.

As part of the High Rate Pond Research Program at the Asian Institute of Technology, HSU (1970) conducted research into the individual effects of depth (10 to 18 in.), detention period (0.5 to 1.5 days), and areal BOD loading (200 to 400 lb/acre day) on pond performance. Separation of effects was accomplished by complete control of influent sewage concentration, flow and pond depth. Twenty seven pond conditions were evaluated using the experimental ponds and dosing apparatus shown in Figure 13. Although the ranges of levels studied apply to high rate pond operation, trends of effects are applicable to facultative ponds. Figure 14 describes the efficiencies of BOD removal in the experimental ponds and it can be seen that depth exhibited no significant influence. As might be expected from the preceding discussion, detention period had only a slight effect on BOD removal, there being no significant difference in removals between 1 and 1.5 days but some loss in efficiency at 0.5 days. As previously observed through the analysis of facultative pond operation, areal BOD loading had by far the strongest influence. Choice of pond depth should be guided by such considerations as mosquito prevention and protection of the lower anaerobic layer, and not BOD removal. It is generally considered that three to five feet is an appropriate facultative pond depth range for tropical conditions.

Influence of Temperature

Areal BOD loading in lb/acre day has been used as an important empirical pond design criterion for decades. Choice of a particular design value has depended upon field experience in any particular location. The effect of temperature on feasible loading rates has not been throughly investigated and applied to design standards on a world wide scale. A review of the literature pertaining to both temperate and tropical climates has revealed that considerably higher loading rates are possible than are currently being used. 'Climate' is herein defined in simplified but quantitative form as the mean monthly ambient temperature, representing both pond water temperature and incident solar radiation. Reported data were analysed in terms of mean monthly ambient temperatures which were readily obtained from ENVIRONMENTAL SCIENCES AND SERVICE ADMINISTRATION (1967).

Primary facultative pond failure is considered to occur when the pond reverts to an anaerobic state throughout the day at all depths. The data were analyzed to reveal maximum feasible pond loading up to the point of failure at any given temperature of operation, the results being shown in Figure 15. In some instances the month of pond

Figure 13. - Experimental Oxidation Ponds used in the study of Depth, Detention Period and Areal BOD Loading Effects.

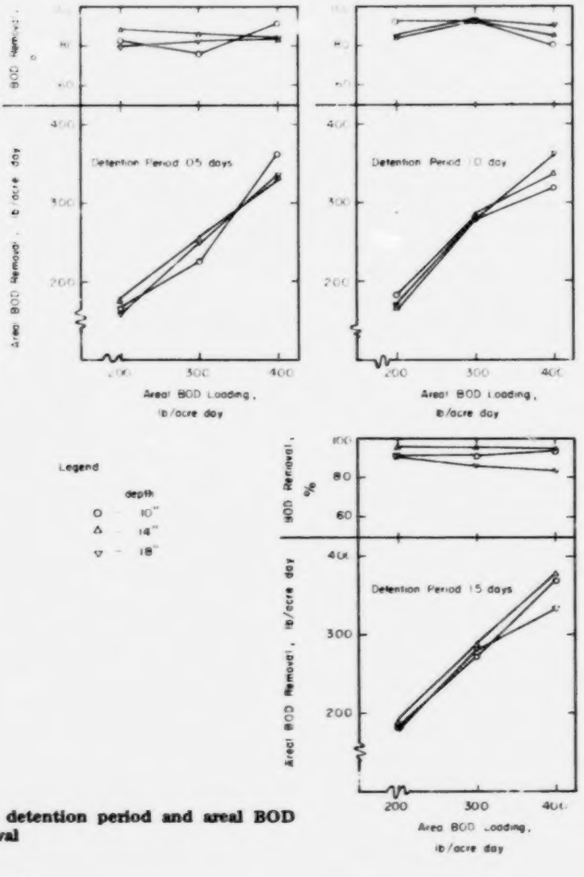

Figure 14. - Effects of depth, detention period and areal BOD loading on pond BOD removal

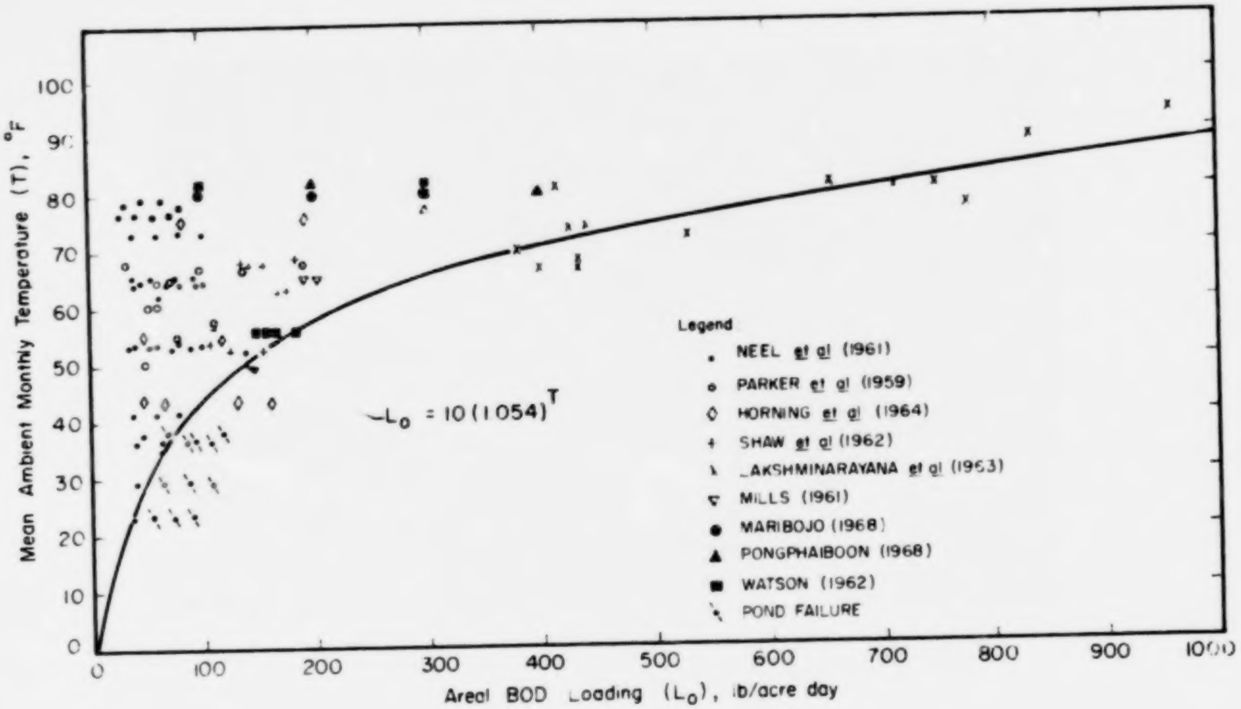

Figure 15. - **Feasible facultative pond loading as a function of temperature**

operation was not given in the literature, however, if the pond was reported to have operated successfully throughout the year the minimum mean monthly temperature was taken as the pertinant operating temperature. An appropriate mathematical model was designed to represent boundary conditions as the point of pond failure as shown in Figure 15. This equation describes the relationship between possible areal BOD loading and ambient monthly mean temperature in the range 20-90° F:

$$L_o = 10(1.054)^T \qquad (9)$$

where, L_o = Areal BOD loading, lb/acre day

T = Ambient mean monthly temperature, °F

This equation appears to resemble the van't Hoff-Arrhenius relationship ($\theta = 1.054$). At high loadings, tropical ponds often tend to maintain a relatively shallow algal-laden aerobic surface layer. Provided this situation is stable, these ponds achieve high rates of biodegredation in the lower anaerobic layers while odours are prevented from escaping through the aerobic surface which acts as a protective layer. This situation is desirable in view of the high stabilization rates achieved by contact anaerobic stabilization.

The boundary curve of Figure 15 approximates operation when the algal layer is just capable of maintaining aerobic conditions at any given temperature. Under these conditions it is likely that all oxygen produced by algae is utilized in oxidation. It is natural that algal production of oxygen is strongly influenced by temperature and solar radiation as reflected by the monthly mean temperature. In fact, Equation 9 suggests that algal oxygen production follow an Arrhenius type equation such that the oxygen produced by algae under high loadings increases by a factor of 1.42 for each ten degree (° F) rise in temperature. It should be emphasized that although Equation 9 may be of the Arrhenius form it is empirical, based on actual data collected in the field, and as such is of considerable value in design. Use of this relationship and Equation 2 in pond design permits estimation of allowable areal loading and expected BOD removal for any given location.

On the basis that the most efficient pond is one which removes the greatest quantity of BOD per unit area, ponds should be loaded to their maximum capacity as indicated by equation 9 for efficient land use and reduced capital cost. By maintaining these rates on primary, secondary and tertiary facultative ponds, subsurface anaerobic conditions will foster high rates of degredation. This implies that ponds should be decreased in surface area along a series of staged facultative ponds and is an extension of the conclusion arrived at for operation of anaerobic ponds in series.

CONCLUSIONS

Anaerobic Ponds

1. Areal BOD loading is the variable having greatest influence on areal BOD removal in anaerobic ponds, with the effects of influent pH and detention period being less significant.
2. The adoption in tropical countries of the maximum areal BOD loading which will permit pond pH to remain above 6.0 when raw waste is fed to anaerobic ponds will result in greatest areal BOD removal.

3. Maintaining maximum areal BOD loading, within the capabilities of the anaerobic system, on each stage of anaerobic pond in a series system will give the highest average areal BOD removal and most efficient land use.
4. Recirculation of a facultative pond effluent containing dissolved oxygen is likely to be effective in suppressing odours in anaerobic pond operation in the tropics but will result in a decrease in overall areal BOD removal.

Facultative Ponds

5. A survey of pond operation in tropical and temperate zones has indicated that the areal BOD removal (L_r, lb/acre day) in primary facultative ponds may be estimated through knowledge of areal BOD loading (L_a, lb/acre day) by the formula:

$$L_r = 9.23 + 0.725\ L_o$$

Standard Error of Estimate 14.9
lb/acre day

6. Within normal operating ranges, detention period and depth have little influence on percentage or areal BOD removal in a primary facultative pond.
7. Most economic primary facultative pond operation is achieved by applying maximum areal BOD loading up to the point of pond failure.
8. Excessive pond loadings result in failure through reversion to completely anaerobic conditions. Maximum feasible pond BOD loadings are related to environmental conditions as reflected by the mean monthly air temperature (T, °F) during operation at any given location according to the equation:

$$L_a = 10(1.054)^T$$

Anaerobic-Facultative Pond Systems

9. To achieve most efficient land use, each stage of anaerobic and facultative pond in a series system should be loaded at the maximum possible areal BOD rate. This will result in decreasing pond areas along the series system.

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SURVIVAL OF ENTERIC BACTERIA AND VIRUSES IN MUNICIPAL SEWAGE LAGOONS

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Durham, New Hampshire 03824

There is limited information in the literature on the fate or survival of enteric bacteria and viruses in oxidation pond systems and on the presence and numbers of these organisms in effluents from such ponds. The majority of the reports published to date have been concerned chiefly with coliform reduction or die-off. Okun (1) reported 99.99 percent reduction of coliforms in each of five experimental ponds when loaded with up to 100 pounds of BOD per acre per day. Drew (2) found maturation ponds to reduce *Escherichia coli* 99.6 percent in summer and 96.8 percent in winter. Coetzee and Fourie (3) report a 99.98 percent reduction of *E. coli* in stabilization ponds and because of the apparent ease with which it is eliminated, they consider this organism may not be an infallible indicator of pathogenic organisms in such pond effluents. This point of view was also expressed by Malchow-Moller, *et al* (4) who considered *Streptococcus faecalis* and *Clostridium welchii* more reliable indicators of pathogens than *E. coli* in the systems they studied. Gann, *et al* (5) found coliform reduction closely associated with BOD removal indicating that the coliforms are removed because of their inability to compete successively for nutrients. Geldreich, *et al* (6), in a study of raw sewage and effluent from a waste stabilization pond located at a state prison dairy farm, reported a reduction in coliform bacterial density from a low of 85.9 percent in the winter to a high of 94.4 percent in the autumn. Fecal coliform reductions were greater than 87.9 percent while reductions of fecal streptococci were 97.4 percent or more.

There have been few reports in the literature on the die-off of pathogens in oxidation ponds. Costzee and Fourie (3) found the total reduction of *Salmonella typhi* in the effluent from two stabilization ponds operated in a series with detention periods of 20 days to be 99.5 percent. The reduction of *E. coli* was 99.98 percent during this same period. In heavily polluted water they found *S. typhi* more resistant to die-off than *E. coli*.

Using *Staphylococcus aureus* and *Serratia marcescens* as test organisms in dialysis tubing immersed in the aerobic zone of a lagoon with a detention time of 6 to 7 weeks, Conley, *et al* (7) found an almost complete die-off of these organisms in 36 to 48 hr. This was not the case in the deeper anaerobic zone.

In regard to the elimination of viruses, Malherbe, *et al* (8) using a model system of four ponds in series, concluded that the biological processes which improve the effluent chemically and bacteriologically were unlikely to alter the virus content so that virus removal depended on their adsorption to static surfaces, exposure to rays of the sun, or a retention beyond the normal survival time of the virus. Christie (9) found that field lagoons reduced the titre levels of polio virus from 10^5 to less than 10^3 units per ml.

With the increased use of lagoons or oxidation ponds as an economical method of domestic waste disposal in this country, further information is needed regarding the potential pollutant hazards such pond effluents may have on receiving waters. In the present study, the survival of coliforms, fecal coliforms, fecal streptococci, salmonellae, and enteric viruses was determined in oxidation pond systems used by three different communities in New Hampshire for the disposal of domestic wastes. Particular attention was given to the effects of seasonal temperatures and the operation of the ponds in series on the survival of these microorganisms.

MATERIALS AND METHODS

Oxidation ponds.

The oxidation ponds used for these studies included a one-pond system for Community A, 8 acres in area serving a population of 600 families, a three-pond system for Community B, each pond 5 acres in area treating about two million gallons of domestic waste per week, and four-pond system for Community C, each pond approximately 10 acres in area treating eight to ten million gallons of domestic waste per week. All ponds were 5 feet in depth. The ponds for Communities B and C could be operated in series. The final effluent from the Community A system was discharged into a fresh water river and from Community C into an estuary which drained into shellfish growing waters. An aerial photograph of the Community C oxidation ponds with the estuary to the left of Ponds 3 and 4 is shown in Figure A.

Collection and processing of samples.

Samples of raw sewage and pond effluents were collected in sterilized disposable quart containers attached to a sterilizable handle. All samples were transported to the laboratory in an iced container and processed immediately.

The number of coliforms and fecal coliforms in the samples were established using standard membrane filter procedures or the confirmed MPN method using 5 replicates in 3 significant dilutions (10). Standard membrane filter procedures (10) or a plating technique were employed for the enumeration of fecal streptococci using m-enterococcus agar (BBL).

Procedures similar to those employed by Slanetz, *et al* (11) were used for the detection of salmonellae. These organisms were concentrated by filtering 1 liter of the sample through a coarse cotton pad covered with about 0.2 grams of diatomaceous earth (Celite - Johns-Manville product) held in a 100 ml diameter Buchner funnel. After filtration of the sample, the pad was cut in half and one section placed into a 250 ml widemouth bottle containing 100 ml of cystine selenite broth (BBL) plus 60 micrograms of albamycin per ml to inhibit Proteus organisms, and a layer of mineral oil over the surface to inhibit pseudomonads with incubation at 43C. The other half of the pad was placed in a similar bottle containing 100 ml of tetrathionate broth (Difco) plus the albamycin and mineral oil with incubation at 35C. At 24 hr intervals up to 7 days, 0.1-0.2 ml amounts of these cultures were smeared on bismuth sulfite agar (Difco) and brilliant green agar (Difco). Following incubation at 35C, colonies characteristic for salmonellae were identified by the procedures recommended by Sanders, *et al* (12). This involved transferring growth from typical colonies into Sander's broth, incubating cultures at 35C, and subjecting those cultures producing a permenant blue or purple color without pellicle to IMViC and fermentation tests. Finally, cultures giving positive reactions for salmonellae were serologically typed using Poly OA-1 Salmonella antiserum (Difco).

Virus isolations from water were accomplished using gauze pads suspended at the collection site for periods of 3 to 7 days. Fluids and fine particulate matter expressed from pads at pH 8.0 to 8.5 were clarified by light centrifugation and treated overnight with an equal volume of ethyl ether. The subnatant aqueous layer was recovered, clarified by low speed centrifugation, and following loss of ether by evaporation at room temperature, inoculated into cell cultures. Three types of cell cultures were used for each examination: primary monkey kidney (African Green), HEp-2 and a stable monkey kidney cell line, LLCMK$_2$. Cell culture monolayers in plastic flasks were overlain with inocula, a 2 hr adsorption period at room temperature observed, followed by rinsing of monolayers and addition of maintenance media. The cultures were incubated at 36.5C and observed daily for evidence of cytopathogenic effects. All samples were passed twice in each cell culture type before they were considered negative for virus.

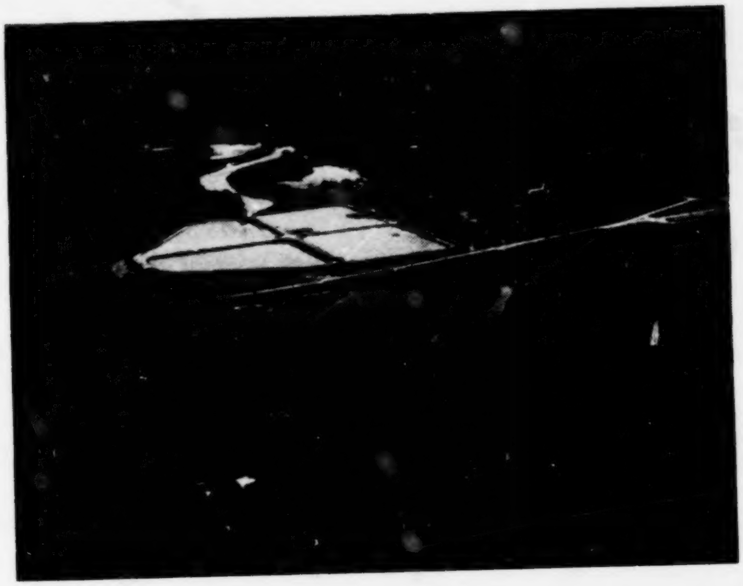
RESULTS

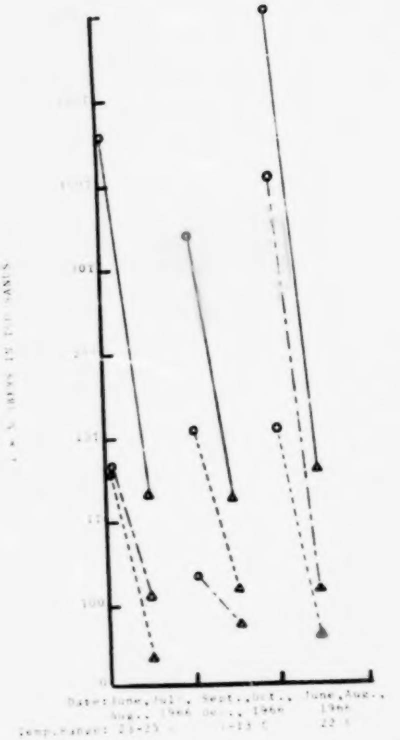
The numbers of indicator bacteria in raw sewage and the numbers or percent surviving in effluents from oxidation ponds used to dispose of domestic wastes from three commumities are presented in Figures 1-6 and Tables 1-6. The temperatures of the sewage or effluents at the time of collection are noted in the figures and tables, and the salmonellae and enteric virus isolations are indicated in the tables.

The survival rates of coliforms, fecal coliforms, and fecal streptococci in the one-pond system used by Community A ranged from 1.5 to 5, 1.6 to 3, and 0.5 to 10 percent respectively based on the numbers of these bacteria detected in the effluents as compared to the numbers in the raw sewage discharged into this pond (Figure 1 and Table 1). The numbers of coliforms in the effluents from this pond which was discharged directly into a small river, ranged from 1,400 to 6,000 per ml. The highest percent survival of the fecal coliforms and fecal streptococci occurred during the low temperature season, September through December. Salmonellae were isolated from 2 out of 5 of the effluent samples and enteric viruses from 2 out of 2 effluent samples tested from this pond.

The results of survival studies of indicator organisms, salmonellae, and enteric viruses in the oxidation pond systems used by Community B are tabulated in Figures 2 and 3 and Tables 2 and 3. The first tests were made when two ponds were being used in series. As the data presented in Figures 2 and Table 2 indicate, the die-off rate of the indicator bacteria in Pond 1 was similar to that reported for the Community A one pond system. However, the numbers of indicator bacteria in effluents from Pond 2 were somewhat greater than the numbers present in the Pond 1 effluents. During the period of this study, there was only a small discharge of effluent from Pond 2 due to seepage of the liquid waste through the gravel bottom of this pond providing a stagnant condition which apparently was suitable for some growth of the indicator bacteria.

Figure A. Aerial photograph of Community C oxidation ponds.

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Aug., 1966 Nov., 1966 1966
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Figure 1. — Numbers of indicator bacteria per ml in sewage and pond effluents based on geometric mean counts of weekly samples during test periods. Community A - One pond. , coliforms; , fecal coliforms; , fecal streptococci; O, sewage; △, Pond 1 effluent.

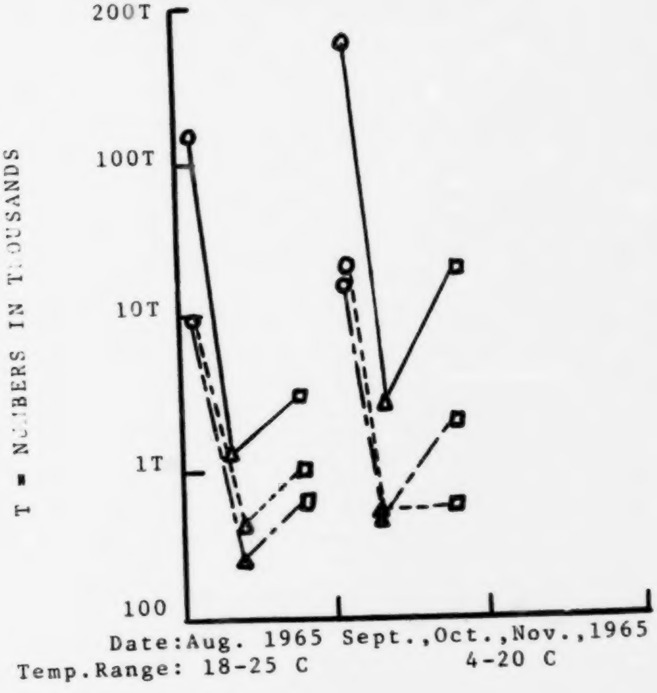

Figure 2. — Numbers of indicator bacteria per ml in sewage and pond effluents based on geometric mean counts of weekly samples during test periods. Community B - Two ponds in series. , coliforms; , , fecal coliforms; , fecal streptococci; O, sewage; △, Pond 1; □, Pond 2 effluents.

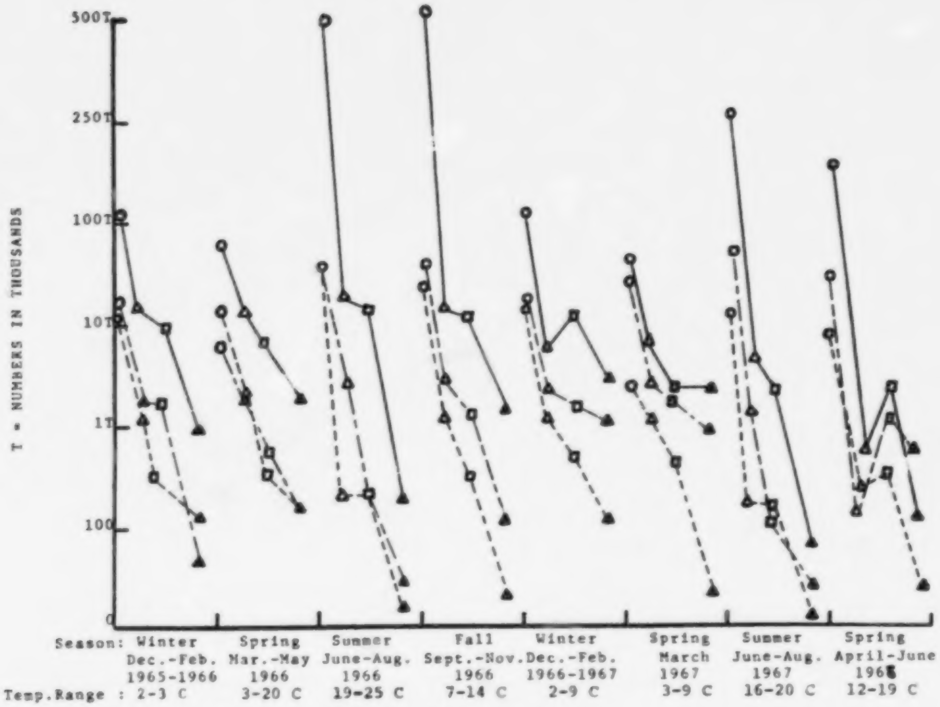

Figure 3. -- Numbers of indicator bacteria per ml in sewage and pond effluents based on geometric mean counts of weekly samples during test periods. Community B - Three ponds in series. , coliforms; , , fecal coliforms; , fecal streptococci; 0, sewage; △, Pond 1; □, Pond 2; ▲, Pond 3 3ffluents.

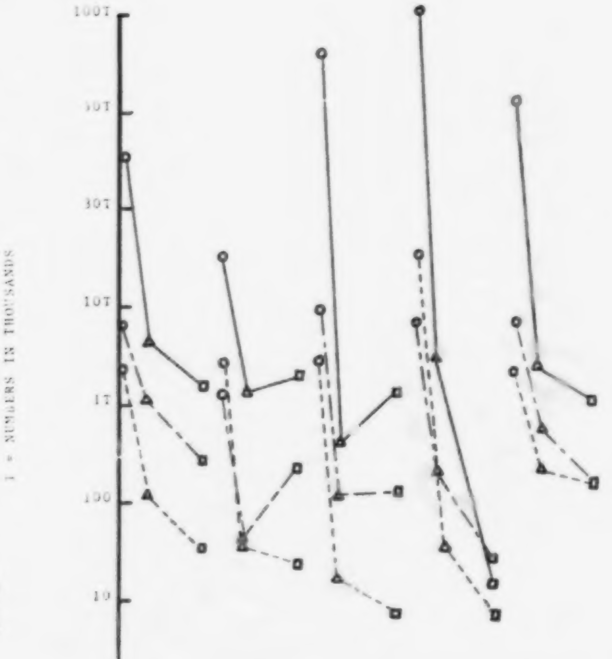

Figure 4. — Numbers of indicator bacteria per ml in sewage and pond effluents based on geometric mean counts of weekly samples during test periods. Community C - Two ponds in series. , coliforms; , fecal coliforms; , fecal streptococci;; 0, raw sewage; △, Pond 1; □, Pond 2 effluents.

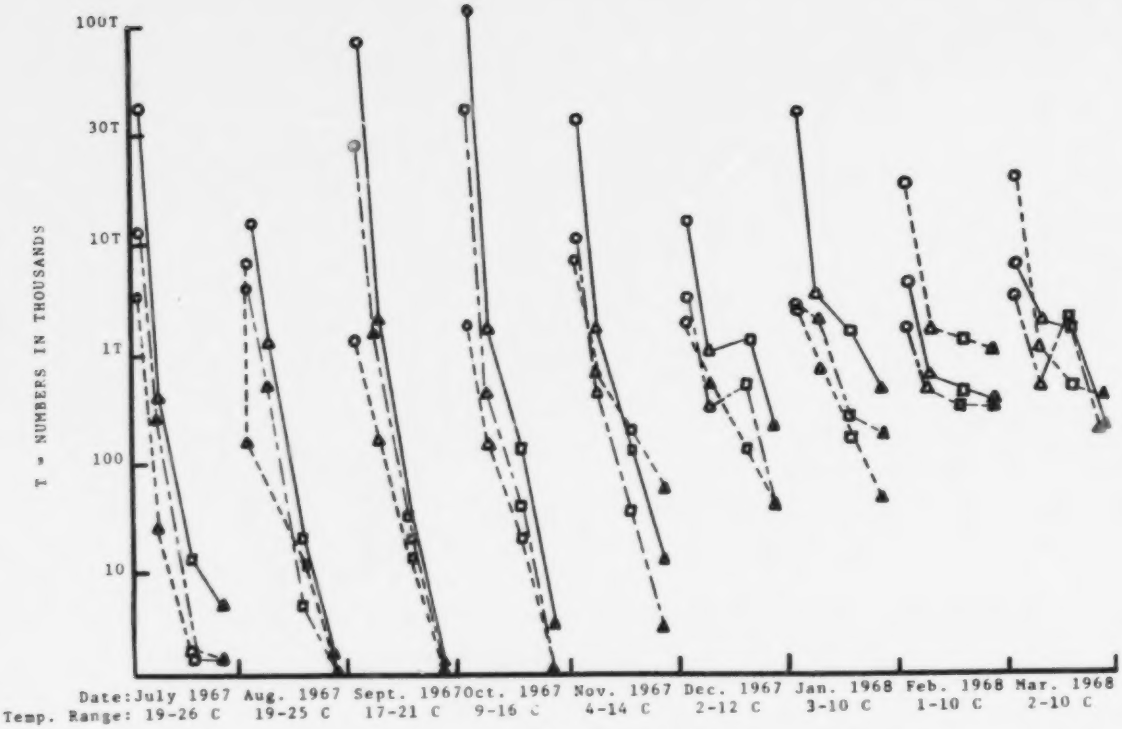

Figure 5. — Number of indicator bacteria per ml in sewage and pond effluents based on geometric mean counts of weekly samples during test periods. Community C - Three ponds in series. , coliforms; , ,, fecal coliforms; , fecal streptococci; 0, sewage; △, Pond 1 effluents; □, Pond 2 effluents; ▲, Pond 3 effluents.

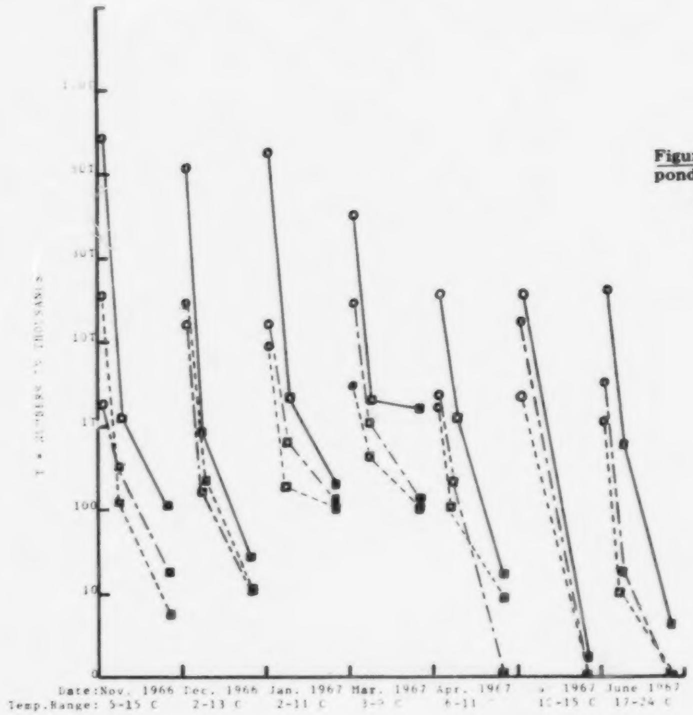

Figure 6. — Numbers of indicator bacteria per ml in sewage and pond effluents based on geometric mean counts of weekly samples during test periods. Community C - Four ponds in series. ,, coliforms; ,, fecal coliforms; , fecal streptococci; 0, sewage; □, Pond 2; ■, Pond 4 effluents.

Table 1

Survival of Indicator Bacteria, Salmonellae, and Enteric Viruses in a One Oxidation Pond System — Community A

Date	Temperature Range	Coliforms	Fecal Coliforms	Fecal Streptococci	Isolation of: Salmonellae	Virus
6/66 - 8/66	23-25C	1.5*	2.0*	0.5*	1/4**	2/2**
9/66 - 12/66	7-13C	2.6	3.0	10.0	3/3	2/2
6/67 - 8/67	22-25C	5.0	1.6	0.6	1/1	-[t]

* Percent survival of these bacteria in pond effluents based on numbers originally present in the raw sewage. Percentages represent geometric mean of membrane filter counts for the total number of weekly samples collected during the test periods.

** Number of times isolated from samples tested.

t No test.

Table 2

Survival of Indicator Bacteria, Salmonellae, and Enteric Viruses In Two Oxidation Ponds Operated in Series — Community B

Date	Temperature Range	Coliforms	Fecal Coliforms	Fecal Streptococci	Isolation of: Salmonella	Virus
			Pond 1			
8/65	18-25C	1.3*	4.6*	7.0*	-[t]	-[t]
9/65-11/65	20-4C	2.6	3.8	3.5	1/10**	4/4**
			Pond 2			
8/65	18-25C	2.7	8.3	9.0	-	-
9/65-11/65	20-4C	11.4	12.9	3.4	4/10	4/4

* Percent survival of these bacteria in pond effluents based on numbers originally present in the raw sewage. Percentages represent geometric mean of membrane filter counts for the total number of weekly samples collected during the test periods.

t No test.

** Number of times isolated from samples tested.

Table 3

Survival of Indicator Bacteria, Salmonellae, and Enteric Viruses in Three Oxidation Ponds Operated in Series - Community B

Date	Temperature Range	Coliforms	Fecal Coliforms	Fecal Streptococci	Isolation of Salmonella	Isolation of Virus
			Pond 1			
12/65-2/66	2-3C	9.9*	11.2*	10.5*	2/10**	8/8**
3/66-5/66	3-20C	15.2	41.2	32.0	6/14	14/14
6/66-8/66	19-25C	6.4	8.6	0.8	8/11	5/5
9/66-11/66	7-14C	3.4	2.1	2.1	2/3	1/2
12/66-2/67	2-9C	5.6	18.4	9.9	3/4	0/1
6/67-8/67	16-26C	2.7	2.3	3.6	5/6	-t
			Pond 2			
12/65-2/66	2-3C	5.9	10.7	6.1	2/10	10/10
3/66-5/66	3-20C	12.1	9.8	4.6	4/14	12/12
6/66-8/66	19-25C	5.3	8.7	0.2	8/11	5/5
9/66-11/66	7-14C	2.2	1.2	1.2	2/3	1/1
12/66-2/67	2-9C	8.3	7.8	4.2	2/4	1/2
6/67-8/67	16-26C	1.1	0.2	2.5	5/6	-
			Pond 3			
12/65-2/66	2-3C	0.6	0.3	1.6	0/10	-
3/66-5/66	3-20C	0.5	0.1	2.6	10/14	12/12
6/66-8/66	19-25C	0.1	0.1	0.1	4/11	-
9/66-11/66	7-14C	0.3	0.3	0.3	1/3	-
12/66-2/67	2-9C	3.2	4.0	1.1	1/4	0/1
6/67-8/67	16-26C	0.03	0.06	0.09	2/4	-

* Percent survival of these bacteria in pond effluents based on numbers originally present in the raw sewage. Percentages represent geometric mean of membrane filter counts for the total number of weekly samples collected during the test periods.

** Number of times isolated from samples tested.

t No test.

Table 4

Survival of Indicator Bacteria, Salmonellae and Enteric Viruses in Two Oxidation Ponds Operated in Series — Community C

Date	Temperature Range	Coliforms	Fecal Coliforms	Fecal Streptococci	Isolation of: Salmonella	Isolation of: Virus
			Pond 1			
6/66	17-25C	15.0*	13.7*	2.5*	3/3**	1/1**
7/66	18-25C	5.5	5.0	1.4	4/4	1/2
8/66	19-24C	0.8	1.3	0.5	2/5	1/2
9/66	19-24C	0.5	3.8	0.3	2/2	2/2
10/66	13-19C	8.3	10.0	3.0	4/6	3/3
			Pond 2			
6/66	17-25C	4.8	6.4	1.2	2/3	1/1
7/66	18-25C	14.4	27.6	1.0	2/3	1/2
8/66	19-24C	1.6	1.1	0.5	3/5	2/2
9/66	19-24C	0.02	0.4	0.04	2/2	1/1
10/66	13-19C	2.0	2.7	2.1	8/8	3/3

* Percent survival of these bacteria in pond effluents based on numbers originally present in the raw sewage. Percentages represent geometric mean of membrane filter counts for the total number of weekly samples collected during the test periods.

** Number of times isolated from samples tested.

Table 5

Survival of Indicator Bacteria, Salmonellae and Enteric Viruses in Three Oxidation Ponds Operated in Series — Community C

Date	Temperature Range	Coliforms	Fecal Coliforms	Fecal Streptococci	Isolation of: Salmonella	Isolation of: Virus
Pond 1						
7/67	19-26C	1.7*	4.0*	0.75*	-t	-t
8/67	19-25C	10.1	12.2	2.5	-	-
9/67	17-21C	3.9	8.5	9.5	-	-
10/67	9-16C	2.2	1.8	9.0	-	-
11/67	4-14C	4.9	6.3	9.2	-	-
12/67	2-12C	7.9	14.5	31.2	-	-
1/68	3-10C	17.0	85.0	50.5	-	-
2/68	1-10C	24.5	7.4	13.5	-	-
3/68	2-10C	34.0	10.0	6.4	-	-
Pond 2						
7/67	19-26C	0.03	0.02	0.05	-	-
8/67	19-25C	0.24	0.1	1.0	-	-
9/67	17-21C	0.06	0.3	0.7	-	-
10/67	9-16C	0.17	0.2	1.3	-	-
11/67	4-14C	0.8	0.5	1.4	-	-
12/67	2-12C	12.0	16.0	7.8	-	-
1/68	3-10C	4.3	12.0	38.3	-	-
2/68	1-10C	8.9	9.0	6.9	-	-
3/68	2-10C	25.3	22.8	4.2	-	-
Pond 3						
7/67	19-26C	0.02	0.02	0.04	1/4**	-
8/67	19-25C	0.02	0.003	0.003	0/4	-
9/67	17-21C	0.002	0.001	0.001	0/2	-
10/67	9-16C	0.005	0.004	0.08	0/5	3/3**
11/67	4-14C	0.05	0.05	1.1	1/4	2/2
12/67	2-12C	2.0	1.6	2.8	1/3	-
1/68	3-10C	1.6	6.0	17.1	1/4	1/1
2/68	1-10C	10.0	10.0	5.2	0/4	4/4
3/68	2-10C	3.7	5.1	3.9	0/4	-

* Percent survival of these bacteria in pond effluents based on numbers originally present in the raw sewage. Percentages represent geometric mean of MPN counts for coliforms and plate counts for fecal streptococci of weekly samples collected during the test periods.

t No test.

** Number of times isolated from samples tested.

Table 6

Survival of Indicator Bacteria and Salmonellae in Four Oxidation Ponds Operated in Series — Community C (Data for Ponds 2 and 4 only)

Date	Temperature Range	Coliforms	Fecal Coliforms	Fecal Streptococci	Isolation of: Salmonellae
Pond 2					
11/66	5-15C	1.8*	2.1*	1.4*	2/4**
12/66	2-13C	1.9	1.6	3.2	2/4
1/67	2-11C	6.0	6.7	3.4	1/4
3/67	3-9C	7.7	7.3	1.2	2/3
4/67	6-11C	5.6	8.1	5.4	0/1
5/67	10-15C	-t	-t	-t	-t
6/67	17-24C	3.0	0.81	0.69	1/4
Pond 4					
11/66	5-15C	0.15	0.12	0.05	0/4
12/66	2-13C	0.12	0.12	0.05	0/4
1/67	2-11C	0.5	1.1	2.2	0/4
3/67	3-9C	5.3	1.0	4.6	2/3
4/67	6-11C	0.05	0.08	0.4	1/3
5/67	10-15C	0.01	0.0	0.04	0/5
6/67	17-24C	0.02	0.05	0.1	0/4

* Percent survival of these bacteria in pond effluents based on numbers originally present in the raw sewage. Percentages represent geometric mean of membrane filter counts for the total number of weekly samples collected during the test periods.

** Number of times isolated from samples tested.

t No test.

Salmonellae were isolated from 1 out of 10 samples from Pond 1, and 4 out of 10 samples from Pond 2 respectively. Enteric viruses were isolated from all of the four samples tested from each of the ponds. The results of tests for indicator bacteria and enteric pathogens when the oxidation ponds for Community B were operated in a three pond series are indicated in Figures 3 and Table 3 over an 18-month period. It is apparent from this data that there was some decrease in the numbers of the indicator bacteria in effluents from Pond 2 as compared to Pond 1, and that a considerable further die-off occurred in Pond 3. It can also be noted that the greatest die-off of these organisms occurred during the summer months. For example, for Pond 3 effluents the percentage survival of coliforms, fecal coliforms, and fecal streptococci was 3.2, 4.0 and 1.1 for the test period December 1, 1966 to February 28, 1967 when the temperatures of the ponds ranged from 2-9C, and 0.03, 0.06 and 0.09 respectively for the period June 1, 1969 to August 31, 1969, when the temperatures ranged from 16 to 26C. Salmonellae and enteric virus isolations were still detected in effluent samples from all three ponds.

Studies were initiated on effluents from the oxidation ponds provided for the disposal of the domestic waste from Community C shortly after they were put into operation. Figure 4 and Table 4 give the numbers of indicator bacteria and the enteric pathogen isolations for sewage and effluent samples when the system was operated on the basis of two ponds in series over a 5-month period. The results were similar to those reported for the Community B two-pond system. In general, there was no major increased die-off of these organisms in Pond 2 over that occurring in Pond 1. During the month of July there was an increase in the number of coliforms in the Pond 2 effluent compared to the Pond 1 effluent. Isolations of salmonellae and enteric viruses were similar to both pond effluents.

The results obtained when this oxidation pond system was operated using three ponds in series are tabulated in Figure 5 and Table 5 based on tests made over a 9-month period. The percentage survival of the indicator bacteria in effluents from Ponds 1 and 2 are similar to those reported previously although there was an appreciable additional die-off in Pond 2 during the period July 1, to November 30, 1967. Again there was greater survival of these organisms during December to March when the temperatures were 2-21C. The almost total die-off of indicator bacteria and salmonellae was quite dramatic in Pond 3 effluents during the period July 1, to October 30, 1967. For example, based on weekly tests made during the month of September when the temperature ranged from 19-25C, the percent survival of coliforms, fecal coliforms, and fecal streptococci was 0.002, 0.001, and 0.001 respectively. No salmonellae were isolated from 2 samples tested during this period. It is also of interest to note that for samples tested during August, when the percent survival of coliforms, fecal coliforms, and fecal streptococci was 0.005, 0.004, and 0.08 respectively, no salmonellae were isolated from 5 weekly samples tested but enteric viruses were isolated from all 3 effluents examined. It might be noted here that Poliomyelitis Type I, II, and III viruses were isolated most frequently from the various pond effluents tested. Isolation of Coxsackievirus B^4 and ECHO virus Type 6 were also accomplished. It should also be pointed out that procedures used for the detection of both salmonellae and enteric viruses involved the concentration of these organisms by filtration of the samples for salmonellae and suspension of gauze pads at the point of sewage or effluent discharge for the viruses. Thus, no information was provided as to the numbers of these organisms in the raw sewage or the pond effluents.

Finally, the results of studies on the survival of indicator bacteria and salmonellae are presented in Figure 6 and Table 6 when these ponds were operated in a four pond series for an 8-month period. These data were based on tests of effluents from Ponds 2 and 4 only. The Pond 4 effluent was discharged directly into an estuary without further treatment. It is evident from this data that only very small numbers of indicator bacteria were present in the Pond 4 effluents. The greatest die-off in the ponds can again be noted during the warmer season when, for example in Pond 4 effluents, 8 coliforms, no fecal coliforms, and an average of less than 1 fecal streptococcus per ml were detected in the samples tested during the month of June. However, it is also interesting to note that even during the winter period of December and January only small numbers of indicator bacteria had survived in Pond 4 as compared to Pond 3 effluents as reflected in the data presented in Figure 5 and Table 5. Salmonellae were not detected in effluents from Pond 4 tested during the months of November, December, January, May and June. It should be noted that the ponds under study froze over during late December, January, and February, thus altering their oxidation status.

During these studies, a limited attempt was made to establish antibiotic factors that might relate to the die-off of the indicator bacteria in the pond systems. Tests for the presence and concentration of coliform and fecal streptococci bacteriophages in the effluents indicated that their numbers decreased during the warm months and increased during the winter season. *Bdellovibrio* organisms could only be isolated from surface mud surrounding the ponds and not from the pond effluents. The major non-enteric bacterial flora that developed in the Community C oxidation ponds consisted of the genera Flavobacter, Achromobacter, and Pseudomonas. When *E. coli* and *Streptococcus faecalis* were inoculated into broth filtrates of several hundred isolates represented in these genera, no toxic effects could be demonstrated. In many of the filtrates, these indicator bacteria actually increased in numbers. Thus, bacteriophages, *Bdellovibrio* and the bacteria chiefly involved in the digestion of organic matter in the ponds did not appear to have a major toxic action on the indicator bacteria. A number of membrane filter filtrates of the pond effluents did show a toxic effect for coliforms and fecal streptococci but this was not consistent with high or low die-off rates in the ponds.

SUMMARY

The die-off of coliforms, fecal coliforms, and fecal streptococci in domestic wastes duscharged into the 5 to 10 acre oxidation ponds studied ranged from 95 to 99 percent when these ponds were operated as one or as two ponds in series at temperatures of 17 to 26C. The percentage survival of these organisms was much greater during the winter season in such ponds when the temperature ranged from 1 to 10C. Salmonellae and enteric viruses were isolated from a majority of the effluent samples tested from these ponds during all seasons of the year.

When the ponds were operated in a series of three or four ponds, very few viable indicator bacteria remained in the effluents from the last pond in the series when the temperature of the ponds ranged from 10 to 26C. Under such conditions, counts as low as 2 per ml or less were obtained for coliforms, fecal coliforms, and fecal streptococci. Salmonellae could be detected in only 1 out of 24 effluent samples tested from the third or fourth pond in the series during the summer periods. However, enteric viruses were isolated from a number of these effluent samples. The numbers of indicator bacteria were appreciably higher and the enteric pathogen isolations more frequent in effluents from the ponds during the winter seasons. Thus, it must be recognized that effluents from oxidation ponds may not be free from bacterial or viral pathogens and such effluents may create a health hazard if discharged without further treatment into waters used for bathing or shellfish growing areas.

ACKNOWLEDGMENT

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BACTERIA REMOVAL IN OXIDATION PONDS

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INTRODUCTION

With the passage of the Water Quality Act of 1965 (P.L. 89-234), and in particular the water quality standards provision, increased attention has been focused on bacterial quality of the nation's waters. The 1968 *Water Quality Criteria* report of the National Technical Advisory Committee (1) suggested various bacterial levels for a number of different water uses. For primary contact recreation, the fecal colifrom (FC) level recommended was a monthly log mean (geometric mean) of 200/100 ml. For general recreation, an FC indicator density averaging 2,000/100 ml with a maximum not to exceed 4,000/100 ml was recommended. Other water uses called for different bacterial criteria. The appropriateness of these suggested levels is not the subject of this paper, but their existence does serve to illustrate the emphasis being placed on health-related bacterial indicators.

To those of us working for regulatory agencies, sources of pathogenic bacteria are of prime importance. At the top of the list we usually find the point source discharges of sanitary wastes from urban areas. In many of our states we also note the feedlots and grazing lands are also significant pathogenic bacteria sources.

The bacterial quality of effluents produced by conventional primary or secondary waste treatment processes has been documented in the literature over the years. Less known, however, are the capabilities of oxidation ponds for bacterial removal effectiveness. Some studies have been reported (2), (3), (4), (5), which indicated total and/or fecal coliform densities in pond effluents. Additional pond study data are also available but bacterial quality is frequently mentioned only incidentally. Many of the reported investigations have shown bacterial reductions of at least 99 percent. Effluent densities may still be quite high even with this reduction, and, unless sufficient dilution capacity is available in receiving waters, water quality standards may be violated.

The Federal Water Quality Administration (FWQA), Southeast Region, in Atlanta recognized that the large number of oxidation ponds which have been constructed in the past decade could materially and perhaps detrimentally affect water quality. These ponds continue to be built and it was felt that more information on bacterial removal was needed in order to determine if design improvements would be necessary. Bacterial data for ponds in the Southeast were generally nonexistent.

Beginning in January 1968, and continuing at intervals through February 1970, FWQA staff from the Technical Services Program of the Southeast Water Laboratory conducted studies at oxidation pond sites located in Georgia and Mississippi. Initially, small ponds (less than 10 acres) in the vicinity of Athens, Georgia, all but one of which were single cell types, were investigated. In the summer of 1969, the study shifted to the southern half of Mississippi, the site of the largest municipal oxidation ponds in the Southeast. Some of these large ponds were single cell, some multicell operated in parallel, and some multicell operated in series. Throughout the investigation, primary attention was placed on bacterial quality of influent and effluent. In some cases, organic loading was determined, and at all pond sites, hydraulic loading was measured.

FACILITIES STUDIED

In all, nine municipal or subdivision type oxidation ponds were studied. The design and operational characteristics of each are summarized in Table I. The schematic layouts for each pond are shown in Figures 1 through 5. Care was taken to study the types of ponds representative of those found in the Southeastern U. S. As might be expected, most are small subdivision ponds, the type which most frequently produces the typical pond problems of odors, emergent weed growth, and generally poor maintenance. Localized pollution has resulted in many small streams which receive subdivision pond effluents.

Of the nine ponds listed in Table I, only those in Hattiesburg and Laurel, Mississippi, received appreciable amounts of industrial waste. In both cases, the industrial waste was from poultry processing, and the primary effect on the oxidation ponds was an increase in organic loading. The larger ponds in Mississippi were the best maintained. This was especially true at Jackson, where operators were on duty throughout the daylight hours, and where operational flexibility was also greatest. There the operators were able to recycle poor effluent from any of the six cells, and could also adjust the operational scheme to any one of several combinations.

Rather than showing detention time in Table I, a volume displacement time figure based on the average influent flow is listed for each pond. True detention time is not easily determined and was not determined for any of the ponds studied. Some dye tracer work was performed this year at the Green Acres and Cedar Creek ponds, but it was directed toward determining short-circuiting problems rather than evaluating actual detention time. At the outset of our investigation, it was felt that the existence of severe short-circuiting problems, particularly in smaller ponds, could be the dominant factor affecting effluent bacterial quality. Only limited work at two ponds was performed with dye tracers. The listing of volume displacement time in Table I provides some insight into relative detention times but should be considered together with other factors such as inlet and outlet design and placement, pond configuration, number of cells, and flow rate variability.

For over half the ponds, some measure of organic loading (as indicated by biochemical oxygen demand techniques) was made. The loading at Winder, Georgia, was estimated but was probably accurate. Pretreatment in the first cell at Winder, an aerated lagoon, was depended upon to reduce the relatively high organic load. In the Laurel, Mississippi, ponds, the higher BOD_5 loadings were attributable to poultry wastes.

BACTERIOLOGICAL STUDY METHODS

Grab samples of influent and effluent were collected several times per day at each oxidation pond studied.[1/] On occasion, composite samples were obtained, but in all cases the samples were returned for analysis within four hours of collection. Samples were iced following collection. The predominate use of grab techniques permitted determination of instantaneous bacterial quality, which is important to know if densities vary appreciably with time as is frequently the case in pond effluents.

Indicator coliform measurements were used to reflect bacterial quality. Both total and fecal coliform densities were determined and in most cases by both multiple tube (MPN) and membrane filter (MF) techniques. The fecal coliform procedure used for the MPN dilution technique was that outlined in *Standard Methods* (6). This procedure employs the standard presumptive test using Lauryl Tryptose broth followed by fecal coliform confirmation using EC medium at an elevated temperature of 44.5°C ± 0.5°C (waterbath).

The fecal coliform MF procedure used was that of Geldreich et al (7), which employs M-FC broth and an incubation temperature of 44.5°C ± 0.5°C (waterbath).

Samples were examined qualitatively for the presence of Salmonella organisms in a few instances.

Seasonal variation studies were made at the Green Acres and Cedar Creek ponds in the Athens, Georgia, area.

Over 600 samples were examined for total and fecal coliform content during the investigation. Sampling was spread over a two-year period.

BACTERIOLOGICAL RESULTS

General

For a thorough understanding of the large amount of bacterial data collected for the nine oxidation ponds, the reader is referred to Table II, which summarizes these data. Arithmetic averages are reported in lieu of geometric means or median values in order to conform to the results generally found in the literature. The geometric mean is actually the preferred method of recording central tendency since it minimizes the effect of inordinately high or low values, e.g. those frequently reported using MPN

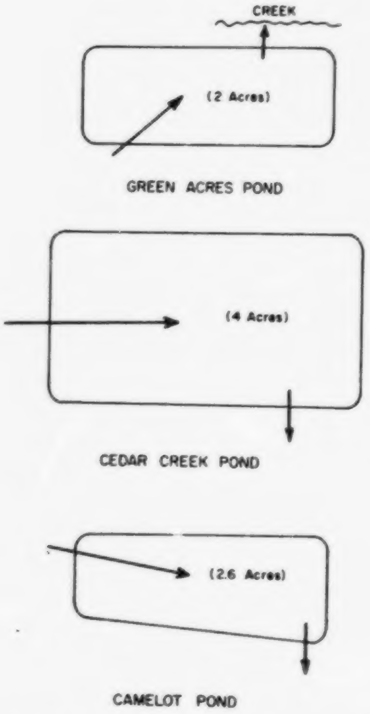
[1/] No samples from within the ponds were examined for bacterial quality.

TABLE I

OXIDATION POND CHARACTERISTICS

LOCATION	DEPTH (feet)	SURFACE AREA (acres)	NUMBER OF CELLS	AVERAGE INFLUENT FLOW (mgd)	VOLUME DISPLACEMENT TIME (days)	ORGANIC LOADING (lb BOD_5/ac/day)	REMARKS
Green Acres Subdivision, Athens, Georgia	4	2	1	0.030	85	22	120 houses. Flow highly variable.
Camelot Subdivision Athens, Georgia	4	2.6	1	0.015	225	30*	69 houses.
Cedar Creek Subdivision Athens, Georgia	3	4	1	0.035	110	17*	100 houses.
Winder, Georgia	4	6	2	0.35	22	100*	First cell is a 0.5 acre aerated lagoon.
Jefferson, Georgia	4	6	1	0.1	75	30*	
Jackson, Mississippi	4	90	6	1.1	110	22	Serves residential area. Series operation.
Hattiesburg, Mississippi	5	360	4 (of approx. equal size)	1.5(2 cells) 2.5(2 cells)	200 120	36 35	Pairs of cells operated in parallel. Two cells receive 1.5 mgd (including poultry waste) and other two receive 2.5 mgd.
Laurel, Mississippi:							
north pond	5	100	1	1.5	110	44	Receives poultry processing waste.
south pond	5	100	1	2.0	80	95	Receives poultry processing waste.

*estimated.

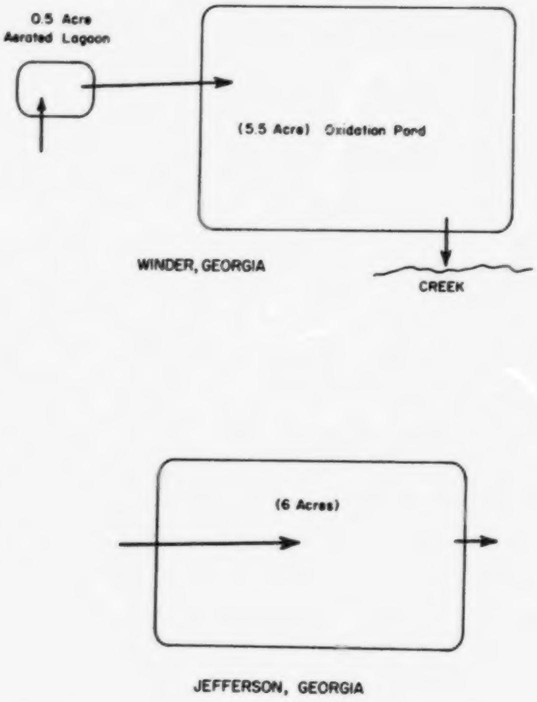


GREEN ACRES POND

CEDAR CREEK POND

CAMELOT POND

FIGURE 1. POND LAYOUTS, ATHENS, GA. AREA



WINDER, GEORGIA

JEFFERSON, GEORGIA

FIGURE 2. POND LAYOUTS, WINDOR, GA. JEFFERSON, GA.

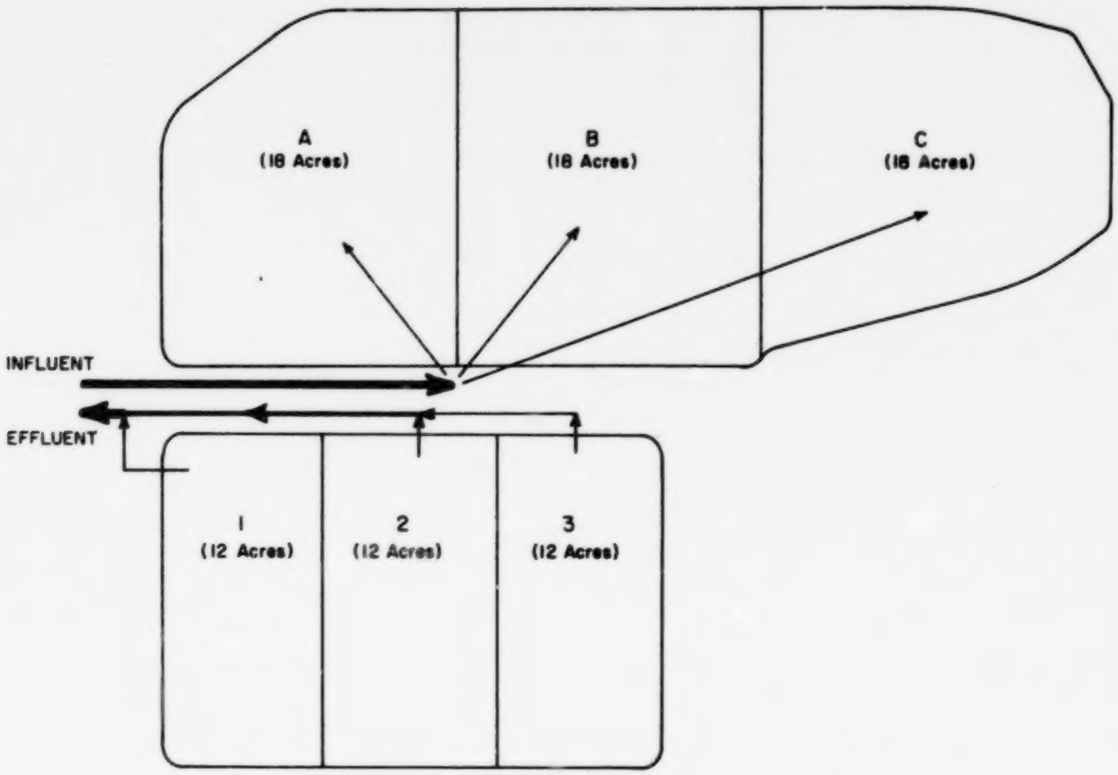

A
(18 Acres)
B
(18 Acres)
C
(18 Acres)
INFLUENT
EFFLUENT
1
(12 Acres)
2
(12 Acres)
3
(12 Acres)

FIGURE 3. JACKSON, MISSISSIPPI OXIDATION PONDS FLOW SCHEME: INFLUENT SPLIT EQUALLY INTO PRIMARY CELLS A, B, AND C. EFFLUENT FROM PRIMARY CELLS ENTER SECONDARY CELLS 1, 2, AND 3 (A TO 1, C TO 2, AND B TO 3).

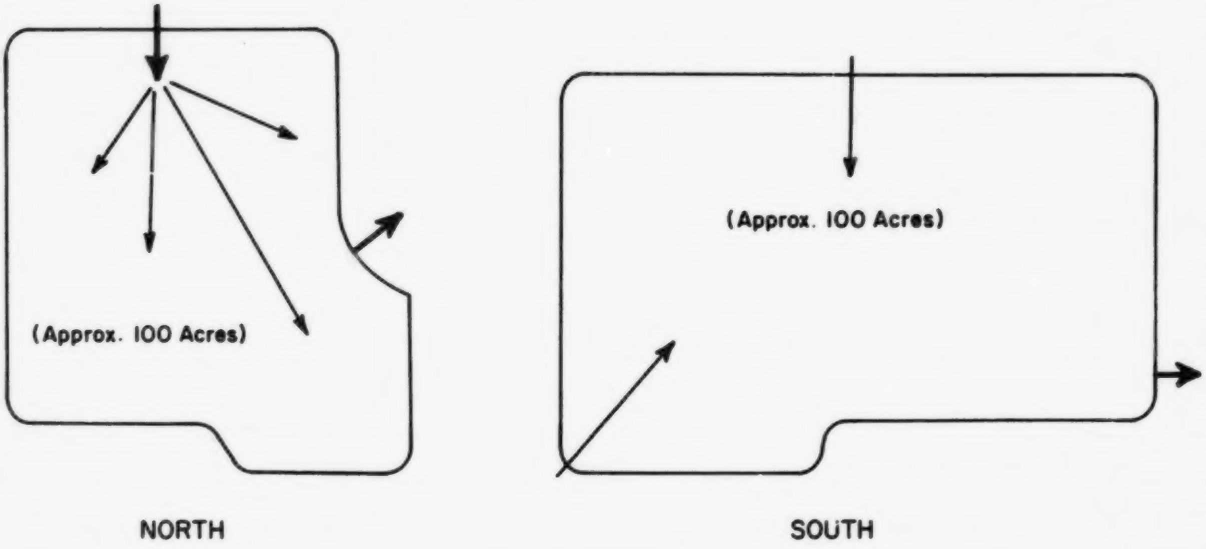

(Approx. 100 Acres)
(Approx. 100 Acres)
NORTH
SOUTH

FIGURE 4. LAUREL, MISS. OXIDATION PONDS

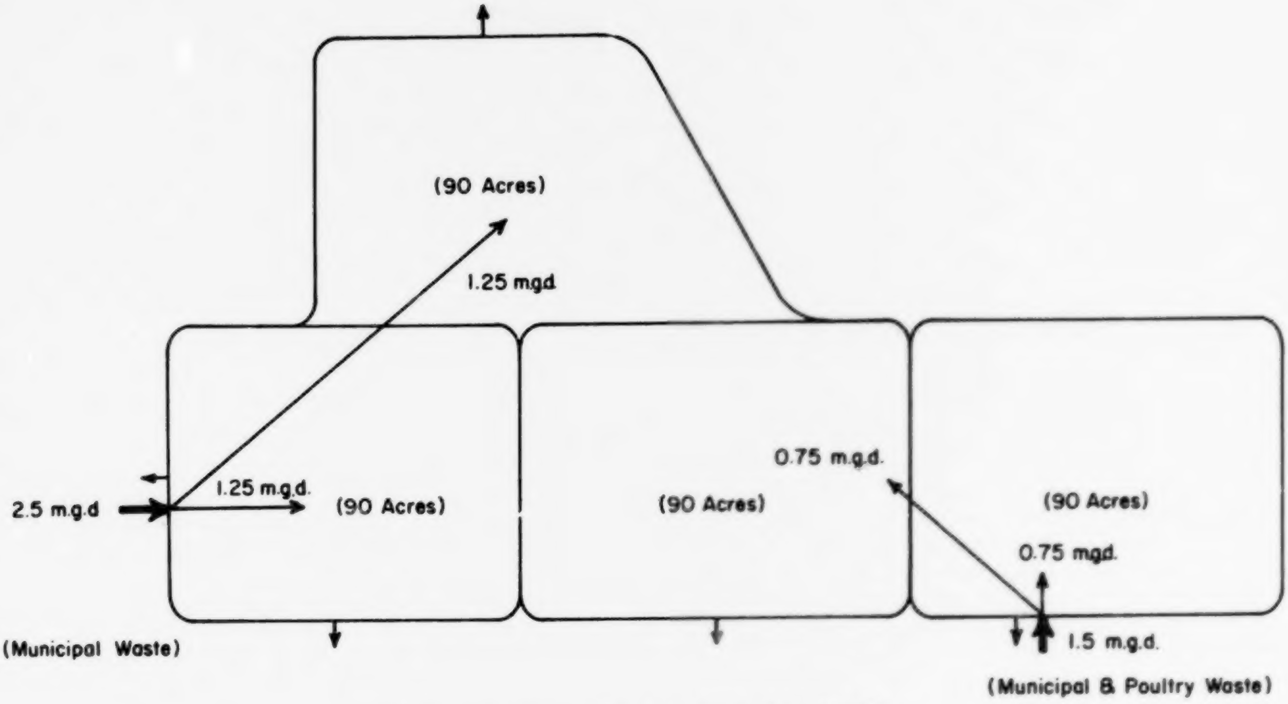

(90 Acres)
1.25 m.g.d.
2.5 m.g.d
1.25 m.g.d.
(90 Acres)
(90 Acres)
0.75 m.g.d.
(90 Acres)
0.75 m.g.d.
1.5 m.g.d.
(Municipal Waste)
(Municipal & Poultry Waste)

FIGURE 5. HATTIESBURG, MISS, OXIDATION PONDS

TABLE II

SUMMARY OF BACTERIOLOGICAL RESULTS

POND LOCATION		TOTAL COLIFORM/100 ml				FECAL COLIFORM/100 ml			
		MPN		MF		MPN		MF	
		Influent	Effluent	Influent	Effluent	Influent	Effluent	Influent	Effluent
Green Acres: (201 Samples*)	Winter Avg.	26,000,000	1,100,000	46,000,000	1,700,000	1,400,000	35,000	2,500,000	87,000
	Summer Avg.	86,000,000	8,100,000	54,000,000	2,900,000	22,000,000	1,000,000	6,500,000	170,000
	Maximum	160,000,000	160,000,000	160,000,000	5,700,000	160,000,000	5,400,000	51,000,000	590,000
	Minimum	3,100,000	130,000	2,900,000	490,000	210,000	3,000	130,000	13,000
Camelot: (52 Samples*)	Winter Avg.	91,000,000	840,000	47,000,000	235,000	6,800,000	2,000	3,100,000	4,000
	Maximum	160,000,000	2,400,000	156,000,000	412,000	17,000,000	3,100	9,800,000	11,400
	Minimum	35,000,000	230,000	11,000,000	10,000	310,000	500	160,000	1,900
Cedar Creek: (101 Samples*)	Winter Avg.	36,000,000	160,000	23,000,000	45,000	3,300,000	9,300	2,100,000	5,700
	Summer Avg.	180,000,000	1,500,000	--	--	44,000,000	200,000	--	--
	Maximum	920,000,000	16,000,000	75,000,000	98,000	350,000,000	1,700,000	8,400,000	10,000
	Minimum	2,300,000	11,000	6,100,000	7,700	490,000	2,000	560,000	700
Winder, Ga.: (60 Samples*)	Winter Avg.	26,800,000	1,480,000	12,300,000	1,100,000	2,400,000	85,000	1,800,000	51,000
	Maximum	92,000,000	3,500,000	31,000,000	8,100,000	4,900,000	230,000	4,900,000	73,000
	Minimum	7,000,000	490,000	2,100,000	400,000	790,000	33,000	100,000	30,000
Jefferson, Ga.: (16 Samples*)	Summer Avg.	110,000,000	283,000	--	--	26,500,000	71,000	--	--
	Maximum	160,000,000	790,000	--	--	92,000,000	230,000	--	--
	Minimum	13,000,000	110,000	--	--	2,300,000	11,000	--	--
Jackson, Miss.: (49 Samples*)	Summer Avg.	253,000,000	374,000	166,000,000	830,000	26,000,000	1,100	29,000,000	470
	Maximum	430,000,000	790,000	246,000,000	1,100,000	79,000,000	2,000	54,000,000	1,000
	Minimum	70,000,000	2,600	63,000,000	550,000	4,900,000	200	4,000,000	40
Hattiesburg, Miss.: (116 Samples*) 1.5 mgd cells	Summer Avg.	100,000,000	715,000	55,000,000	1,130,000	32,200,000	109,800	15,900,000	258,000
	Maximum	240,000,000	5,400,000	175,000,000	6,000,000	110,000,000	2,400,000	37,000,000	3,500,000
	Minimum	14,000,000	23,000	7,000,000	10,000	2,200,000	2,000	2,400,000	2,000
2.5 mgd cells	Summer Avg.	80,100,000	473,000	45,000,000	326,000	24,500,000	128,000	19,600,000	73,000
	Maximum	240,000,000	24,000,000	99,000,000	23,000,000	92,000,000	490,000	41,000,000	570,000
	Minimum	7,900,000	12,000	5,000,000	10,000	2,300,000	2,000	2,400,000	2,800
Laurel, Miss.: north pond (23 Samples*)	Summer Avg.	66,400,000	2,402,000	16,000,000	3,490,000	15,000,000	47,000	10,600,000	45,000
	Maximum	240,000,000	5,400,000	63,000,000	6,600,000	31,000,000	80,000	16,000,000	180,000
	Minimum	7,300,000	86,000	11,000,000	160,000	7,900,000	11,000	7,000,000	1,000
south pond (23 Samples*)	Summer Avg.	134,100,000	976,000	41,000,000	914,000	38,300,000	270,000	8,800,000	144,000
	Maximum	920,000,000	3,900,000	130,000,000	1,200,000	350,000,000	700,000	29,000,000	620,000
	Minimum	2,200,000	49,000	1,000,000	46,000	800,000	110,000	1,000,000	5,000

*Number of samples inclusive of all sampling points, i.e., influents and effluents. Some samples were composites.

procedures.1/ Extremely high values affected some of the averages rep rted in Table II.

It is readily apparent from Table II that the two coliform enumerative procedures, MPN and MF, yield differing results. This has been observed in many studies at the Southeast Water Laboratory when comparing MPN index values to actual MF plate count. The MPN index normally has the higher density values. The MF value, however, does fall within the MPN 95 percent confidence interval approximately 80 percent of the time. No further elaboration on this point will be presented here as this is part of a broader study on the same subject. Multiple dilution tube results will be used for most of the following discussion since such results are available for all ponds studied. The membrane filter procedure is actually preferred because, among other reasons, it has shorter incubation period requirements.

For comparative purposes, seasonal rather than overall average results are presented in Table II. The seasonal bacterial data for the Green Acres and Cedar Creek ponds showed that significantly lower influent and effluent fecal and total coliform densities occurred during the winter.

Total Coliform Results

Summer MPN total coliform averages for the pond influents ranged from a low of 66,400,000/100 ml for the north pond at Laurel, Mississippi, to 253,000,000/100 ml at Jackson. The values at Jackson were significantly higher than at other ponds. Effluent average densities for summer studies ranged from 283,000/100 ml at Jefferson, Georgia, to 8,100,000/100 ml in the Green Acres pond.

Winter influent densities were lower and less widely divergent (26,000,000 to 91,000,000/100 ml). For effluents, however, there was considerable difference in bacterial quality. At the Cedar Creek pond, the average total coliform density was 160,000/100 ml, while at Winder the average was 1,480,000/100 ml.

The appreciable spread in effluent density results was expected since many different pond types and organic and hydraulic loadings were represented in the nine ponds studied.

Fecal Coliform Results

From the standpoint of sanitary significance, fecal coliform densities were judged more important than total coliform. For recreationally classified waters, all but one of the states in the FWQA, Southeast Region have adopted fecal coliform criteria for their water quality standards.

A fairly narrow range of average influent FC densities (MPN) were found during warm weather seasonal work. The minimum average was 15,000,000/100 ml at the Laurel, Mississippi, north pond,1/ and the maximum was 44,000,000/100 ml at the Cedar Creek oxidation pond. Effluent averages ranged from an extremely low 1,100/100 ml at the Jackson, Mississippi, series-operated pond to a much higner 1,000,000/100 ml at Green Acres Subdivision.

1/ A more detailed report on this study is being prepared which will present results in terms of arithmetic and geometric means.

1/ Also had lowest total coliform influent average.

In the winter, the influent FC averages were lower than in the summer, as was the trend for the total coliform data. The lowest average was 1,400,000/100 ml (Green Acres) and the highest was 6,800,000/100 ml (Camelot). Effluent FC quality values ranged between 2,000/100 ml at Camelot to 85,000/100 ml at Winder, Georgia. Note that in the Camelot pond the highest average influent FC density produced the best quality effluent from a bacterial standpoint. The highest volume displacement time (Table I) was also found at Camelot.

Additional information which the bacterial removal study showed, and which served to fortify the findings from other field surveys conducted from the Southeast Water Laboratory, was that no assured ratio of total to fecal coliform densities could be ascertained. Reports of such a ratio continue, and only serve to confuse the obvious – that dieoff rate differences, predation, variation in bacterial types within total or fecal groups, etc. will never allow a universal ratio to exist.

Bacterial Removal Percentages

With three exceptions, the removal of both total and fecal coliform organisms in all ponds studied was consistently equal to 99+ percent. Removal of total and fecal coliforms at the Green Acres pond was approximately 92 and 95 percent respectively, and at Winder it was 94 and 96 percent. Fecal coliform removal at the north pond site in Laurel, Mississippi, was 99+ percent but total coliform removal was only 96 percent.

As mentioned previously, the actual total and fecal coliform effluent density is more significant from a water quality control standpoint than the removal effectiveness expressed in percent.

Daily Bacterial Variation

Significant total and fecal coliform density diurnal variations were observed at most ponds studied. In pond influents, both total and fecal coliform densities were generally higher in the morning than during the afternoon hours. No representative pattern could be discerned for effluent bacterial quality. Figure 6 shows the results of an intensive study at the Green Acres pond on June 26, 1968, when both total and fecal counts[1] for influent and effluent declined steadily during the day but with pronounced hour-to-hour variations. Within a 20-minute period, effluent FC counts were found to differ by as many as 70,000 organisms per 100 ml.

The occurrence of an unusually high total or fecal coliform effluent count was observed at many pond sites. In most instances, pond outlet structures were constructed to permit spillover of floating fecal matter. To an extent, this design deficiency was believed to be responsible for inordinately high coliform counts.

Salmonella

All samples taken for 1½ miles downstream from the Green Acres pond were positive for Salmonella.

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[1] Membrane filter technique.

FIGURE 6
HOURLY BACTERIAL VARIATION, GREEN ACRES POND 6/26/68

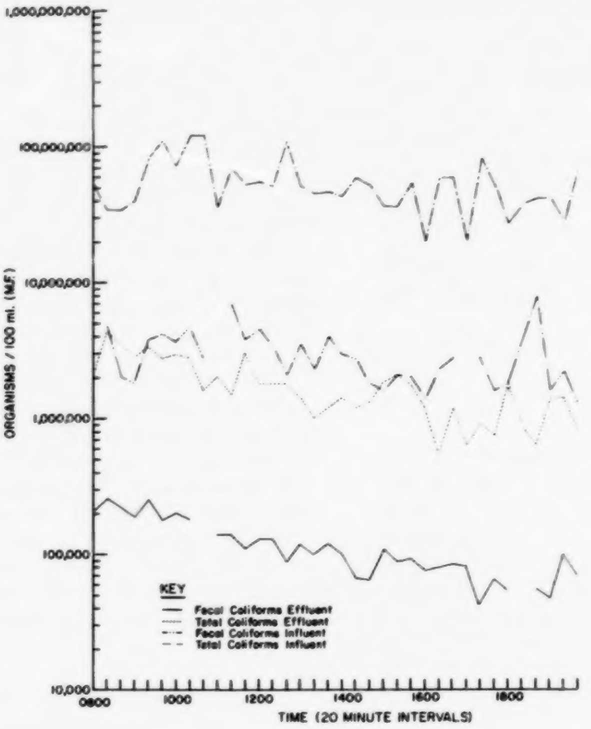


FIGURE 7
EFFECT OF ORGANIC LOAD ON FECAL COLIFORM REMOVAL

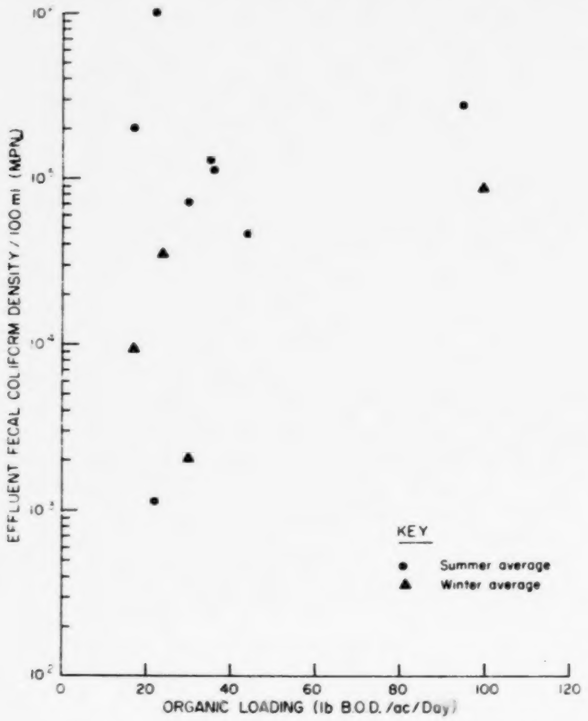
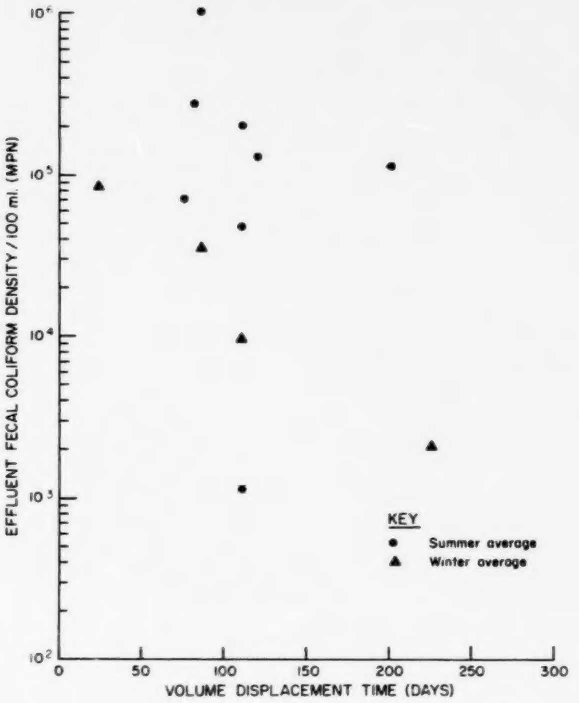


FIGURE 8
EFFECT OF DISPLACEMENT TIME ON FECAL COLIFORM REMOVAL



OTHER POND PERFORMANCE RESULTS

Results of BOD_5 analyses[1] on grab and/or composite samples suggested that removal of organics at several ponds was generally equivalent to that found in conventional secondary treatment plants. Removal efficiencies were 85%, 90%, 90+%, and 90+% at Green Acres, Jackson, Hattiesburg, and Laurel (both ponds) respectively.

To derogate from these acceptable organic removal levels, it was also observed that many floatables, i.e. rubber and plastic articles, fecal matter, greaseballs, paper, etc., passed unimpeded into receiving waters. Imagine if you will the full personal impact of retrieving a 10 pound greaseball while trolling for largemouth bass.

Highly colored (green) effluents also passed from oxidation pond into receiving stream.

DISCUSSION

The most important parameter investigated during the two-year oxidation pond study was effluent fecal coliform density. The preceding section indicates that these FC densities varied appreciably from pond to pond. In order to compare the FC results for different ponds, the average seasonal densities are plotted in Figures 7 and 8 as a function of organic loading and volume displacement time, respectively. The latter is assumed to be an indication of relative, but not necessarily actual, detention time. It should be noted that bacterial densities are plotted on a logarithmic scale in both figures.

Figure 7 presents a scattering of points representing the different ponds which implies little relationship between organic loading (in some cases estimated) and effluent fecal coliform levels. It was not anticipated originally that organic loading would be the most significant factor affecting bacterial removal effectiveness.

When plotted as a function of displacement time (Figure 8), the results were more definitive. A trend toward decreasing densities with increasing displacement time can be interpreted. This was particularly true for the winter values. There were two notable exceptions to this trend and at least for one case, a significant exception. The highest density value of 1,000,000 fecal coliforms/100 ml was for the Green Acres pond. More importantly, the lowest density of 1100/100 ml was at Jackson, Mississippi. The multicelled, 90-acre pond at Jackson was the only true series operation oxidation pond studied[1] and is one of the few such ponds in the southeastern United States. With only slight dilution, the Jackson effluent FC density could meet primary contact recreation criteria in many Southeastern waters. The general appearance of the effluent at Jackson also reflected a much better esthetic quality than for any of the other ponds. Although the waste volume design capacity at Jackson has not been reached

[1] Unfiltered and dark incubation.

[1] The Winder, Georgia, site and series operation but with a small aerated lagoon preceding a polishing pond.

FIGURE 9
DYE, TRACER DATA, CEDAR CREEK POND

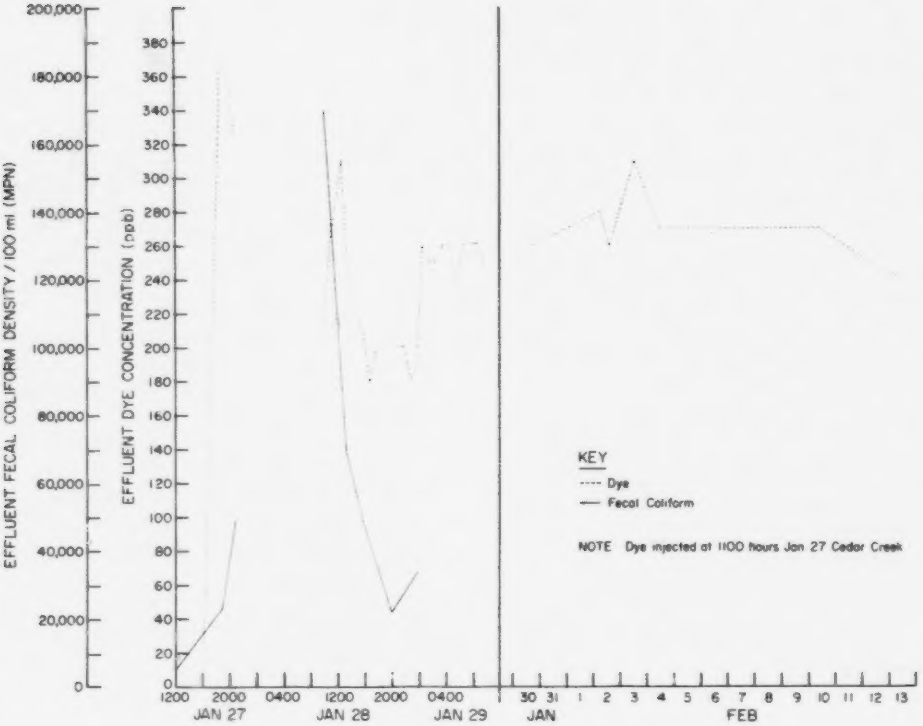
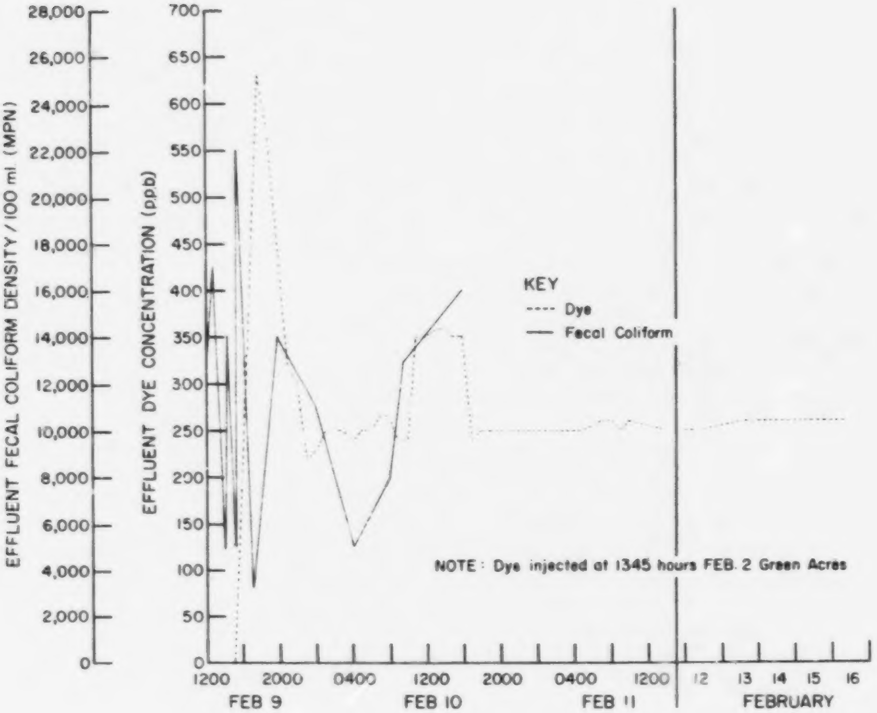


FIGURE 10
DYE TRACER DATA, GREEN ACRES POND



yet, the existing hydraulic and organic loadings were similar to several of the other ponds studied.

Dye Tracer Study

In order to provide better insight into the internal flow patterns in small subdivision oxidation ponds, a dye tracer study was made during January and February 1970. The study was also used to indicate the degree of short-circuiting. Influents of two ponds, Cedar Creek and Green Acres, were injected with Rhodamine WT dye and circulation patterns observed. Effluent bacterial quality (not included in previous discussions) was monitored for approximately two days following each dye injection. Dye concentrations in the effluent were determined for up to two weeks following injection. The basic design of the Green Acres pond had been modified with a small surface aeration unit since the initial and more extensive bacterial studies in 1968. The waste volume at the Cedar Creek pond had almost doubled since 1968. Results of the tracer study are shown in Figures 9 and 10.

At Cedar Creek (Figure 9), dye first appeared at the outlet approximately 6½ hours after injection. Calculated volume displacement time on January 27, 1970, was 80 days. The maximum dye concentration observed was 370 ppb, which was approximately 50 percent higher than the concentration expected with complete mixing of the dye. The degree of short-circuiting for this pond, then, was judged not significant on the date of the study. Different climatological conditions could affect the mixing characteristics more detrimentally.

Visual observations of the dye introduced into the Cedar Creek pond showed that a discrete streak persisted for several hours following injection. Thorough mixing was achieved approximately 36 hours after injection. No attempt was made to determine actual detention time, but it was noted that dye concentrations were maintained at high levels for at least two weeks.

Effluent fecal coliform densities (MPN) at Cedar Creek followed a pronounced diurnal pattern. The minimum density was 5,000/100 ml during midday on January 27 and was a maximum of 170,000/100 ml atabout 10:00 a.m. the following day. The effluent FC counts during the dye study were similar to those recorded in 1968.

Figure 10 shows the dye study results at the Green Acres oxidation pond. Even though an aerator-mixing device was situated centrally in the pond, the effect of some short-circuiting was more pronounced than at Cedar Creek. The maximum dye concentration of 630 ppb appeared approximately four hours after injection. This concentration was roughly 2½ times the calculated complete mix value. As expected, the first dye appearance at Green Acres occurred sooner than at Cedar Creek. Threshold appearance was approximately 1½ hours after injection. Thorough mixing occurred some 27 hours after dye introduction, and no appreciable decline in concentration was noted for the next six days thereafter.

The fecal coliform counts in the Green Acres effluent ranged from a minimum of 3,300 to a maximum of 22,000/100 ml. These densities were lower than those found in 1968, and it was felt, but not proven, that the aerator-mixing device was responsible for improved effluent bacterial quality. Further study should be made on this subject.

The extremely high effluent fecal coliform average at Green Acres in the summer of 1968 has been attributed tentatively to severe short-circuiting. Because of the modification effected since that time, this could not be verified positively.

CONCLUSIONS

The studies reported here were not intended to delve into all the causative factors relating to bacterial survival in oxidation ponds, e.g. available substrate, temperatures, and predation. Rather, the objective was to determine in a general survey fashion the overall bacterial (total and fecal coliform) removal effectiveness of representative pond types in the Southeast. Conclusions which can be drawn from this two-year study are:

A. Effluent fecal coliform densities for many single-celled ponds are of such magnitude as to violate primary contact recreation uses unless appreciable dilution capacity is available in receiving waters. The dilution capacity needed could be in excess of a thousandfold.

B. Series operation produces better bacterial quality effluents by far than single-cell operation. Existing single-cell ponds should be modified and converted to series operation. All new ponds should be designed for series operation.

C. Effluent chlorination could be needed as an alternative to series operation if single-cell pond operation is continued at any pond site. Bacteriological studies would be needed to determine the need for chlorination. Operational practicalities to control the chlorine dosage are also a primary consideration.

D. Organic loading has no appreciable effect on total or fecal coliform effluent quality. True retention time is much more important.

E. Outlet structures at many ponds permit the discharge of floating solids, including fecal solids. These structures must be designed to preclude floating solids loss, although this is not as significant a problem with series operation.

F. Effluent bacterial densities in single cell ponds may vary by an order of magnitude from one moment ot the next.

As Franzmathes stated recently (8), controlled retention rather than uncontrolled average detention must be the goal of design of oxidation ponds.

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CHLORINATION OF WASTE POND EFFLUENTS

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INTRODUCTION

The problem of chlorination whereby disinfection can be dependably obtained in effluents from waste stabilization ponds and lagoons is one causing health officials and water pollution control authorities much difficulty. The effluents of such algal-bacterial systems consist of solutions of various ammonia compounds, a variety of organic substances, inorganic reductants, bacteria, colloids and algae. The chlorine that is introduced into such effluents will interact in a very complicated sequence with these compounds, i.e., it will first react with amino and ammonia compounds, then with other organic reductants, then with unsaturated organic compounds and then eventually certain bacteria and algae. However, indiscriminate chlorination of stabilization pond effluents, which frequently contains large quantities of algae, results in the degradation of algae with the liberation of excessive biochemical oxygen demand. When disposed in receiving waters, this oxygen deficiency could lead to unsuitable conditions for fish life, recreation and water supply. The controlled chlorination of such effluents to destroy pathogenic organisms without degrading the algae is the objective of this study.

LITERATURE REVIEW

Chlorination in Wastewater Treatment

Information from published papers on the reduction of bacteria in sewage by various types of treatment processes have been reported by Imhoff and Fair (1956) and a table adapted from their work is presented (Table 1). These values for bacterial reductions are sometimes used to predict performance of proposed plants where specific data for a particular plant are not known.

A number of discrepancies appear between the plant performance data of Imhoff and Fair and those reported in more recent publications. In many cases more recently reported coliform densities have been shown to increase during the sedimentation process (Camp, 1962; Merrill, 1964). Pomeroy (1955) on the other hand reported the same median coliform densities in raw sewage as in primary effluent. Coliform removals by activated sludge plants reported in Table 1 are less than those indicated by Merrill (1964), whereas greater reductions than those shown in Table 1 have been reported for chlorination of raw sewage by Camp (1962) and treated sewage by Frost (1950) and Hom (1959-60).

A more complete tabulation of coliform removal attained by various authors and in various treatment processes is presented in Figure 1. As is shown, raw domestic sewage coliform densities averaged 5.4×10^7 per 100 ml with extreme values ranging from 10^6 to 10^8 coliforms per 100 ml.

The evidence shown in Figure 1 indicates that there is a systematic correlation between coliform removal and the degree of treatment applied if these data are divided into treatment processes with and without chlorination. In both cases the trend appears to be increasing coliform removal for increasing degrees of treatment. From the figure it is evident that as applied chlorine dosage is increased coliform removals are increased. This increase is evident in raw sewage, and in effluents from primary, trickling filter, activated sludge and stabilization ponds treatment processes. It is evident that holding time (such as is available during holding in stabilization ponds) and chlorination are the best of the available alternatives in coliform bacteria removal.

The Effect of Chlorine Concentration

In evaluating the effect of chlorine compounds for disinfection, it is somewhat conventional to correlate the bacterial kill with the concentration of chlorine remaining after a specified contact time. The term "chlorine demand" is used to describe the difference between the amount of chlorine applied and the concentration remaining at the end of the contact period. In applying the criterion of residual chlorine, it is assumed that the effectiveness of bactericidal action is a function of the chlorine remaining; consequently, the existence of a high chlorine residual is *prima facie* evidence of effective disinfection. It is important to recognize, however, that it is not always necessary to have a high residual to attain a high level of disinfection. In satisfying the chlorine demand, many of the chlorine or hypochlorite molecules may have already achieved the intended objective, viz., the killing or inactivation of pathogens or coliforms. Hence, it is not surprising that chlorine dosages insufficient to produce a chlorine residual may nonetheless be extremely effective in reducing the concentration of indicator organisms.

In describing logarithmic death, utilizing the first order reaction, it is assumed that only the organisms undergo change and that other reactants are constant and are never limiting factors. Such is seldom the case in practical disinfection by chemical means, for the

Table 1 – Bacterial Efficiencies of Conventional Sewage Treatment Processes[a]

Treatment Operation or Process	Bacteria Removed, %
1. Fine screening	10 to 20
2. Chlorination of raw or settled sewage	90 to 95
3. Plain sedimentation	25 to 75
4. Chemical precipitation	40 to 80
5. High-rate trickling filtration including pre- and post-plain sedimentation	80 to 95
6. Standard-rate trickling filtration including pre- and post-plain sedimentation	90 to 95
7. High-rate activated sludge treatment including pre- and post-plain sedimentation	80 to 95
8. Conventional activated sludge treatment including pre- and post-plain sedimentation	90 to 98
9. Intermittent sand filtration	95 to 98
10. Chlorination of biological treated sewage	98 to 99

[a] Imhoff and Fair, *Sewage Treatment*, New York: John Wiley and Sons, Inc. (1956).

Figure 1. Coliform Removal with Increasing Chlorination.

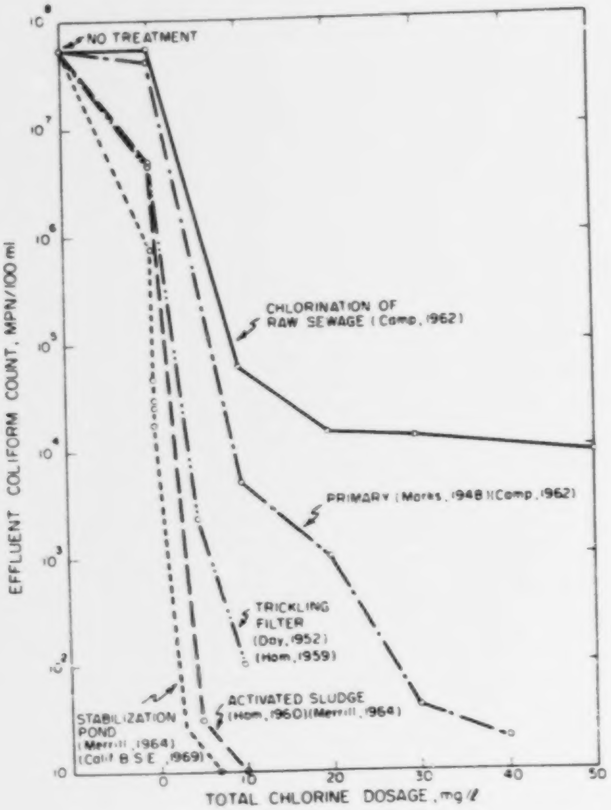


Table 2 –
Typical Stabilization Pond Effluent Characteristics

Algae: Chlorella Spp.	2.6 x 10^6 organisms/ml
Bacteria: Coliform MPN	2.24 x 10^5 organisms/100 ml
BOD, 5 day, with algae	18.1 mg/l
without algae	15.9 mg/l
Nitrogen, Total	20.4 mg/l
Ammonia	13.8 mg/l
Organic	6.6 mg/l
pH	8.1
Suspended Solids	29.6 mg/l
Volatile	25.7 mg/l
Fixed	3.9 mg/l
Total Solids	958.4 mg/l
Volatile	243.3 mg/l
Fixed	717.1 mg/l

disinfectant may be consumed or neutralized before all organisms have been exposed to it. Therefore, chemical disinfection is likely to be a mass action phenomenon. Fair (1954) has observed that the disinfection efficiency with chlorine concentration can be expressed empirically by the equation:

$$C^n t = \text{Constant}$$

in which C is the concentration of chlorine, t is the contact time for a constant percentage of kill, and n is the coefficient of dilution. Values of n greater than one indicate that the efficiency of the disinfectant decrease rapidly with dilution. Values of n less than one indicate that the contact time is of greater importance. When n is equal to one as it is for most chemical disinfectants, concentration and time are equal importance.

MATERIAL AND METHODS

Laboratory tests were run over a period of two years using effluents from the experimental ponds of the City of Concord, California. The composition of the waste was typical of domestic wastewater and was essentially free of industrial wastes (Table 2).

Sampling

Three-day composite samples made up of 500 ml portions were collected at three-hour intervals. The samples were refrigerated after collection. The three one-gallon samples were combined into one sample container from which eight one-liter portions were withdrawn for the chlorination experiments.

Test Procedures

The various determinations were performed according to the eleventh edition (1960) of *Standard Methods for the Examination of Water and Wastewater*, with modifications as noted.

The chlorine demand was determined by the OT test using 30 minutes contact time. Various dosages of chlorine were applied to each sample so that range from 0 to 64 mg/l was obtained, the zero being the control. The samples were mixed immediately after chlorination. At the end of each contact period from 0 to 200 hours, sample portions were withdrawn aseptically into sterile sample bottles which contained sodium thiosulfate solutions to reduce the remaining chlorinous compounds to inactivity. At the time of withdrawal of each portion, chlorine residuals were determined by the OT method. Analyses of coliform bacteria were performed using three dilutions on each sample portion. Analyses of BOD were performed with the alkali-hypochlorite modification so that excess chlorine in the sample is neutralized by the addition of thiosulfate solution.

RESULTS

Results of experiments to test the concomitant effects of chlorine treatment on algae, bacteria, and residual BOD are presented in Figures 2 through 5. It is evident from Figure 2 that increasing doses of applied chlorine decrease the densities of coliform bacteria and increase the degradation of algae after a reaction time of 15 minutes. The differential rates of these two concomitant reactions may be optimized (Figure 3). The plot of the results shows that an optimal point is reached at a dose of applied chlorine of 5.0 mg/l and a residual chlorine of 2.4 mg/l after 15 minutes reaction time.

Similarly, it is evident from Figure 4 that increasing doses of applied chlorine decrease the densities of coliform bacteria and increase the residual BOD after a reaction time of 60 minutes. The differential rate of these two concomitant reactions may be optimized (Figure 5). The results show that an optimal point is reached at a dose of applied chlorine of 3.4 mg/l and a residual chlorine of 1.0 mg/l after 60 minutes reaction time.

Results of concomitant effects of time and concentration of residual chlorine on coliform removal and residual BOD are presented in Figures 6 and 7. Data obtained here are not really adequate for any firm predictions to be made, but do provide empirical relationships by which rational process criteria may be obtained. It is evident from Figure 6 that time-concentration relationships of the form $R^n t = k'$, in which (R) is the residual chlorine concentration and (t) is time of reaction, may be obtained for any given removal of coliform bacteria. Table 3 is a tabulation of (k') constant and dilution coefficient (n) for any given coliform removal of 90 to 99.999 percent.

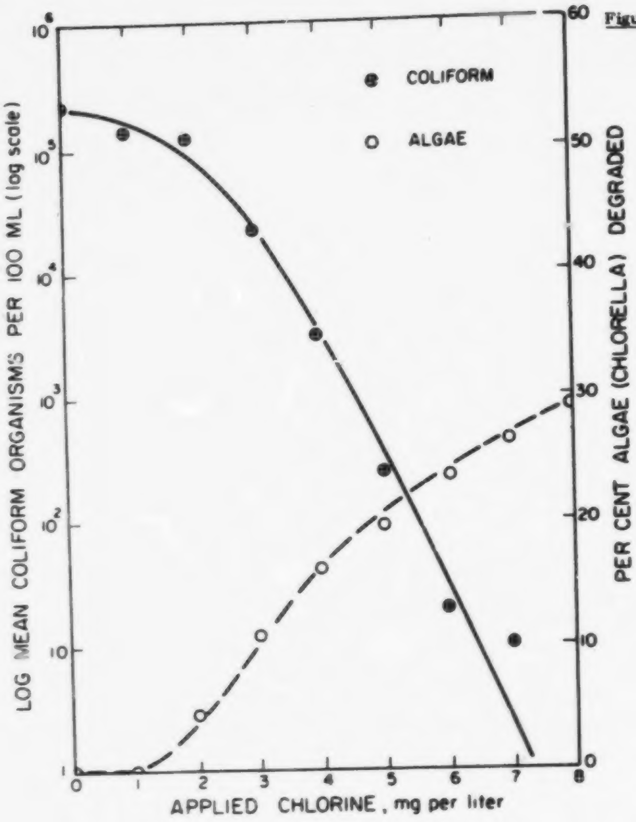


Figure 2. Concomitant Effects of Chlorine Treatment on Algae and Coliform Bacteria.

Figure 3. Optimization of Differential Chlorination After 15 minutes.

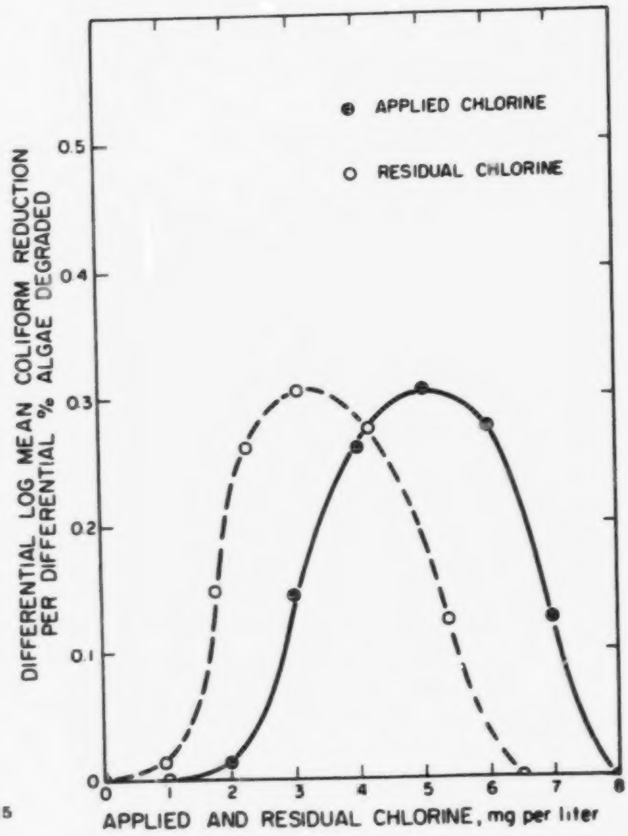
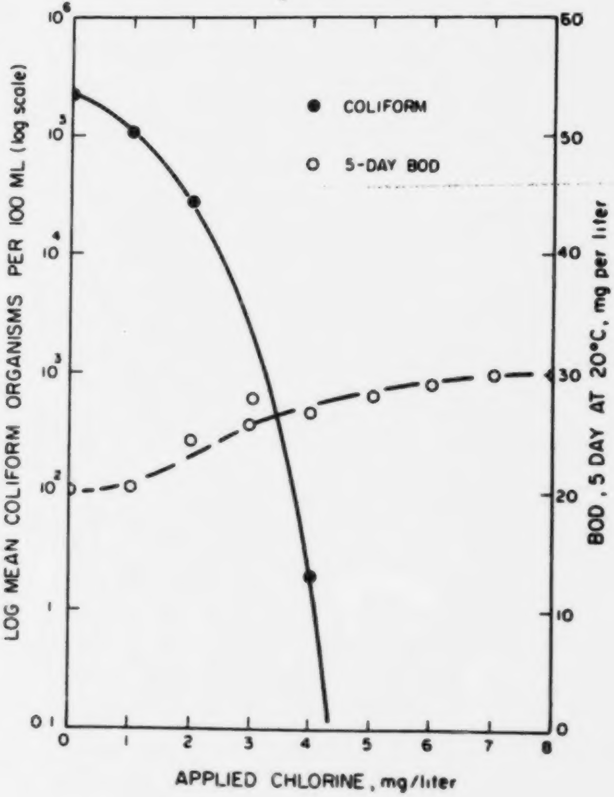


Figure 4. Concomitant Effects of Chlorine Treatment on Coliform Bacteria and Residual BOD.



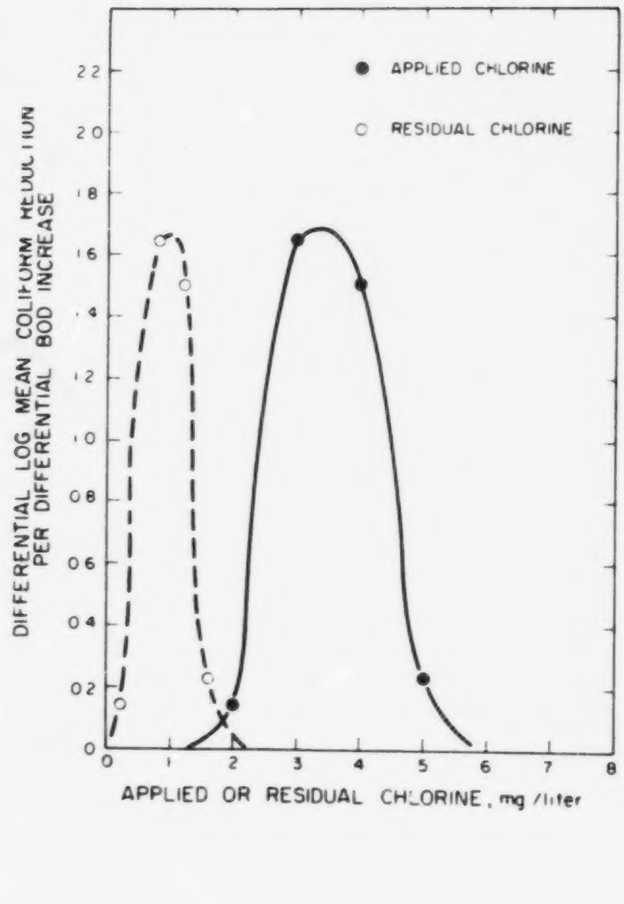


Figure 5. Optimization of Differential Chlorination After 60 Minutes.

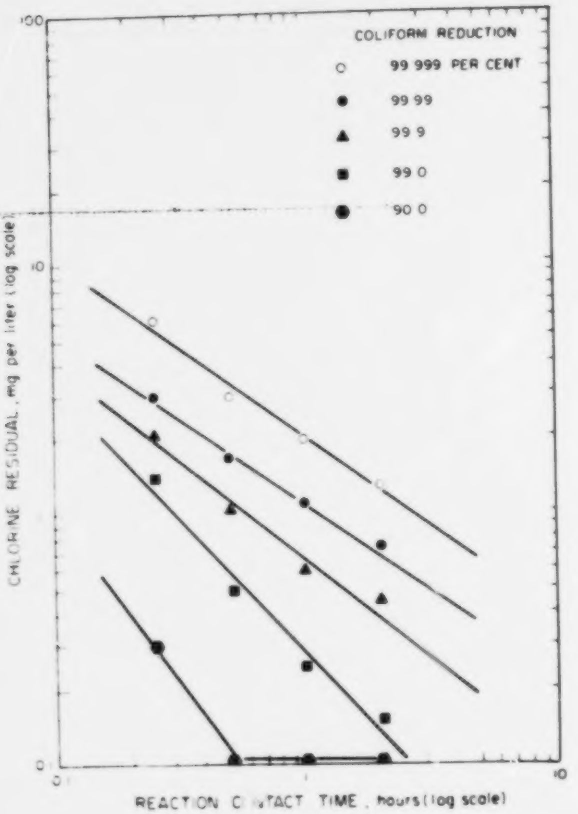


Figure 6. Time-Concentration Effects of Chlorine Treatment on Coliform Reduction.

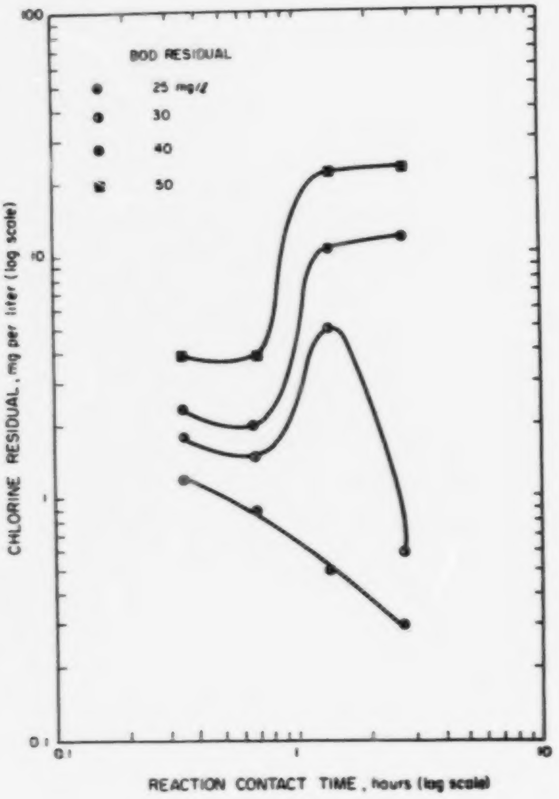


Figure 7. Time-Concentration Effects of Chlorine Treatment on BOD Residual

Table 3 – Rational Process Design Constants

Coliform Reduction Percent	K' MG-HR/L	Dilution Coefficient n
90	0.13	1.09
99	0.28	1.01
99.9	0.53	0.79
99.99	1.18	0.66
99.999	2.45	0.76

Table 4 – Rational Process Design Parameters
Coliform Reduction = 99.99 percent
BOD Residual = 30 Milligram per liter

Reaction Contact Time (t) Minutes	Residual Chlorine (R) MG Per Liter
30.0	1.78
36.0	1.57
42.0	1.43
48.0	1.31
54.0	1.22
60.0	1.13
72.0	0.99
84.0	0.90
96.0	0.82
108.0	0.76
120.0	0.71

Results of time-concentration effects on BOD residual in chlorinated algal-bacterial systems are plotted in Figure 7. From the limited results obtained, it is evident that the time-concentration relationships are not subject to straight line resolution at given levels of BOD residuals. However, the plots of the results do provide boundary conditions of time and concentration of residual chlorine by which rational process design parameters may be obtained. An example of the basic principle of rational process design is given in Table 4. For a given coliform removal of 99.99 percent and an upper level of BOD residual of 30 mg/1, the process design parameters of time and concentration of residual chlorine may be determined from the equation,

$$R^{0.66}\,(t) = 1.16.$$

DISCUSSION

In general, the findings of the experimental studies are that, although stabilization ponds provide adequate removal of BOD to less than 20 mg/1, the results of coliform bacterial densities indicate that disinfection of pond effluents is frequently essential to meet the usual disinfection criteria established for sewage discharges to streams with multiple beneficial uses and relatively high dilution. An example of this is that none of the pond effluent samples collected met the water quality standards established for discharges into the Sacramento River. In the experimental work reported herein the geometric mean coliform MPN of 224,000 organisms per 100 ml exceeded by more than a hundredfold the requirement of a median MPN of 500 per 100 ml and the requirement that no two consecutive samples exceed 7,000 per 100 ml for discharges into the Sacramento River.

The influence of chlorine treatment on algal organisms in samples of pond effluents from the City of Concord experimental ponds indicated that selective chlorination which affects bacteria and algae differently can be achieved under controlled conditions whereby indicator bacterial organisms are destroyed, yet leaving the algae essentially intact (as indicated by BOD determination). Control of time of reaction and concentration of chlorine results in degradation of algae, release of their cellular contents, and thereby a major increase in effluent BOD, defeating several of the purposes of the ponding process. Time is a deciding factor affecting the residual BOD in chlorinated algal-bacterial systems because the production and the reduction of BOD are time dependent. Thus, information on the rates of the reactions or kinetics of chlorine treatment among algal organisms, coliform bacteria, and BOD is essential to a rational application of chlorination processes to pond effluents.

The concomitant effects of applied residual chlorine on algae and coliform bacteria (Figures 2 and 3) suggest that the reaction between chlorine and algae and chlorine and coliform bacteria occur at different rates. Thus optimal points of applied and residual chlorine may be obtained from a plot of differential rates of these two concomitant reactions (Figure 3). The optimal points obtained after reaction time of 15 minutes (applied and residual chlorines of 5.0 mg/1 and 2.4 mg/1, respectively) indicate that maximum removal of coliform bacteria may be attained with a minimum degradation of pond algae. These findings suggest both differences of sensitivities of organisms to chlorine and differences in their rates of reactions with chlorine.

Similar results may be obtained with the concomitant effects of applied and residual chlorine on coliform bacteria and residual BOD (Figures 4 and 5). The optimal points obtained after reaction time of 60 minutes (applied and residual chlorines of 3.4 mg/1 and 1.0 mg/1, respectively) suggest that longer contact time results in a saving of applied chlorine. McKee, et al. (1960), in experiments with chlorination of settled sewage, has also suggested that increasing the chlorine contact time results in a saving of chlorine chemical required for disinfection.

An examination of data from many sources (Butterfield, 1948; Fair, 1948; Moore, 1951; McKee, 1960) shows that the time required to effect a constant percentage of bacterial reduction by means of any disinfectant is dependent on concentration by the following relationship: $C^n t$ = constant. Values of the constant n has been reported by Fair, et al. (1948), for various disinfectants and organisms:

$$HOCl: n = 0.67 \text{ to } 1.6$$

$$NHCl_2: n = 0.75 \text{ to } 1.0$$

A similar relationship to the above may be obtained for concomitant effects of time and concentration of residual chlorine on coliform removal in algal-bacterial systems. Figure 6 is a log-log plot of the relationship: $R^n t = k'$. Values of k' and n for various levels of coliform removal are given in Table 3, in which n varies from 0.76 to 1.09 which is comparable to the values of n reported by Fair, et al. (1948) for monochloramine. Although data obtained here are not really adequate for firm predictive purposes, they do provide a set of empirical relationships by which rational process criteria may be obtained. Thus for any given removal of coliform bacteria desired, the time of reaction and residual chlorine concentration may be determined from Figure 6.

Similar analysis may be made for the effects of time and concentration of residual chlorine on levels of residual BOD (Figure 7). However, log-log plots of time and concentration of residual chlorine at given levels of BOD residual are not subject to straight line resolution. It is evident from the curves plotted in Figure 7 that the reactions of chlorine in algal-bacterial systems produce a residual BOD in a complex manner. From the limited data obtained from the BOD kinetics experiments, it appears that at least two reactions are involved. The first reaction involves the interaction of chlorine and algae by which the algae is damaged in some manner, possibly at the cellular surface. Then the cellular contents are released and become available for degradation in the BOD reaction, and thus BOD is increased.

The second reaction involves the oxidation of a portion of the degradable organics. Thus the degradable organics become less available for degradation in the BOD reaction, and the amount of BOD produced in the first reaction is reduced.

Figure 7 indicates that time of reaction and concentration of residual chlorine have casual effects on the resulting BOD. Although the data for these curves plotted in Figure 7 are not really adequate for firm predictive purposes, they provide an empirical basis for a rational process design of chlorination of pond effluents involving algal-bacterial systems.

With increasingly stringent water pollution control criteria requiring higher quality effluents, and with rising land costs, an increased need for development of dependable disinfection processes is evident. The influence, methodology and prediction of performance of chlorination in algal-bacterial systems developed in this study should make possible rational design for disinfection of effluents containing mixtures of algae and coliform bacteria.

Areas needing further research indicated by this study are numerous. The long-term effects of algal-laden effluents discharging into the receiving waters are not known. Also the possibility for aftergrowths of microorganisms of the coliform group in nutrient-rich receiving waters has not been investigated. Studies of the kinetics of chlorination of effluents containing blue-green algae remain to be made as do studies on the savings of chlorine which may be attained through harvesting of algae contained in effluents prior to chlorination. The effect of chlorine on pathogenic viruses and cysts in algal-bacterial systems has not been studied systematically.

The kinetics of BOD in chlorinated algal-bacterial systems under light and dark conditions should be studied to stimulate depth and seasonal conditions in actual receiving waters. Factors other than time and concentration that affect chlorine disinfection in algal-bacterial systems should be sought and their kinetics investigated. The foregoing factors include light, pH, mixing and the physical, chemical and biological characteristics of the medium. These examples indicate the many areas where fruitful research could be conducted in the disinfection of pond effluents comprising algal-bacterial systems.

SUMMARY AND CONCLUSIONS

Summary

The influence, methodology, and prediction of performance of chlorine disinfection of pond effluents comprising algal-bacterial systems has been investigated in samples of actual pond effluents. Based on the results, the indication is that selective chlorination can be achieved under controlled conditions whereby indicator coliform bacterial organisms are destroyed, yet leaving the algae essentially intact. Control of time of reaction and concentration of chlorine is essential, however, because excessive chlorine releases nutrients from algal cells thereby increasing the effluent BOD, defeating several of the purposes of the ponding process.

Conclusions

1. Disinfection of pond effluents involving algal-bacterial systems is essential to meet the usual disinfection criteria established for wastewater to be discharged to streams which have multiple beneficial uses and relative high dilution.
2. Chlorination of algal-bacterial systems under controlled conditions of reaction time and concentration of residual chlorine is one means of providing dependable disinfection in waste pond effluents.
3. Applied and residual chlorine concentrations can be optimized experimentally thus making it possible to maximize the rate of coliform reduction and minimize the rate of algal degradation.
4. The time-concentration relationships reported herein provide a rational set of process design parameters for chlorine disinfection in algal-bacterial systems.

ACKNOWLEDGEMENT

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EFFECT OF LAGOON EFFLUENT ON A RECEIVING STREAM

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INTRODUCTION

The waste stabilization pond or lagoon is a method of waste treatment that has been widely used only in the past 20 years. In areas where land values are not excessive, the low cost of construction and operation has made lagooning, in one form or another, the method of choice for the stabilization of many different types of waste materials. While careful attention usually is given to construction features, such as operating depth, the slope of the dikes, and inlet and outlet structures, little consideration has been given to the effect of lagoon effluent on receiving streams.

In some areas lagoon contents are discharged only during periods of high water in the receiving stream; but where annual precipitation is equal to or exceeds evaporation, there is a continuous effluent from lagoons. It is the later case that would appear to be of greatest significance to the receiving stream.

Regardless of whether the lagoon is an oxidation pond, a facultative lagoon, or an anaerobic cell followed by an aerobic polishing pond, the effluent characteristically contains a significant concentration of algae. It is these small living plants, so abundant in most lagoons, that give lagoon effluent its unique character and lead to an apparent dichotomy. The planktonic algae discharged from lagoons have the potential for considerable oxygen production but also represent a significant load of energy-rich organic matter to the receiving stream. The fundamental question would appear to be whether the algae in lagoon effluents are of benefit to streams as oxygen generators and a preferred food for aquatic animals or whether they play a detrimental role as an organic pollutant which stimulates excess bacterial activity.

In attempt to answer this question, a study was conducted in the summer of 1967 on Bear Creek, which receives effluent from two facultative municipal lagoons on the North edge of Columbia, Missouri (Figure 1). Bear Creek is an unfortunate little stream which headwaters in the City of Columbia and is subject to severe scour whenever it rains on the city. The portion of the stream considered in this study is 15-20 feet wide with and average depth of about 18 inches. A few of the pools are five to eight feet deep and the depth of the riffles is often less than one inch. The average dry-weather summer flow of Bear Creek above the lagoons is about 0.6 cfs with most of this water coming from the fly ash settling ponds of the Columbia municipal power plant (1). After the stream receives the effluent from the 12.5 acre large lagoon and the 4.8 acre small lagoon, dry weather flow averages about 0.88 cfs and the stream is about 25% lagoon effluent. Groundwater infiltration between the point of effluent of the large lagoon and the termination point of this study, 1900 yards downstream, increases the stream discharge to about 1.05 cfs. At this point the stream is 21.5 percent effluent and lagoon seepage (1).

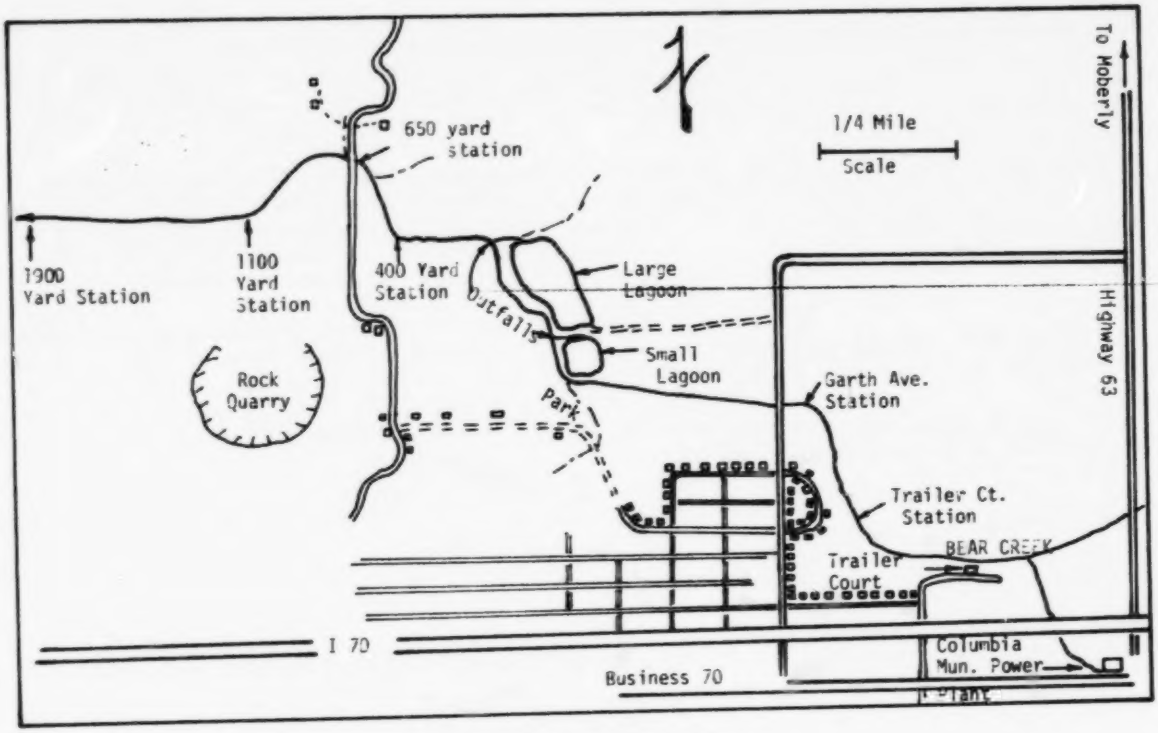

Figure 1. Map of Bear Creek Study section.

CHARACTERIZATION OF LAGOON EFFLUENT

Since lagoon effluents contain myriads of small living organisms, which, in all probability remain alive in BOD bottles for periods longer than five days, the conventional BOD_5 measurement appeared to be a poor choice for estimating the oxygen demand for these effluents. Thus, attempts were made to characterize the effluents by various solids measurements, COD, BOD, and caloric energy (1). In addition, long-term BOD studies were conducted to determine the percentage of the effluent, measured as COD, which was oxidized during a 26 day period.

Determination of total and volatile solids content of the lagoon effluent were adequate, but problems associated with the water of hydration of the clay turbidity in the stream water precluded acquisition of meaningful solids measurements in the stream below the lagoons. Bomb calorimetry measurements of the lagoon effluent gave excellent estimates of the caloric energy content of both the particulate and dissolved portion of the effluent. However, the high clay content in downstream samples caused the combined algae and clay sample to behave in an endothermic fashion in the bomb calorimeter. Thus, while neat, round bricks were made in the bomb calorimeter, negative energy estimates were recorded for the large algal portion of these decidedly green samples. Both COD and BOD appeared to give reasonably accurate estimates of both the effluent and the stream water following addition of the effluent, and it appeared that some combination of these measurements would be required to follow the fate of the effluent downstream.

The next step was to determine the relationship between BOD and COD of the effluents from the two lagoons. The day the first effluent samples were collected for the long-term BOD studies, that from the large lagoon was dark green, algae laden, and dominated by *Aphanizomenon*. In contrast, the effluent from the small lagoon essentially was devoid of algae but contained a tremendous concentration of microcrustacea and rotifers. The COD of the small lagoon was 231 mg 0_2/1 and that of the large lagoon was 251 mg 0_2/1.

Despite a considerable difference in the type of organisms present, it can be seen from Figure 2 that both effluents exhibited similar BOD satisfaction curves. The BOD_5 of both effluents was equal to about 20 percent of the initial COD and by the 26th day BOD was roughly equal to 90 percent of the initial COD. Thus, it appears that the type of microorganisms in lagoon effluent has little effect on the ultimate oxygen demand.

The next step was to evaluate the relative oxygen demand of the particulate and dissolved fractions of the effluent. The effluent samples collected for this determination were dominated by *Aphanizomenon* but contained some green algae and exhibited a total COD of 288 mg 0_2/1 and a COD of the dissolved fraction equal to 91 mg 0_2/1. Determinations of BOD were made at two day intervals for a 26 day period for both the total effluent and the filtered, dissolved fraction of the effluent. The assumption that the dissolved fraction exhibited the same BOD pattern alone as it did in the unfiltered sample allowed calculation of the oxygen demand of the particulate portion of the effluent. The results of these determinations are given in Table 1.

Figure 2. BOD satisfaction curve for large lagoon effluent containing algae and small lagoon effluent containing microcrustacea and rotifers.

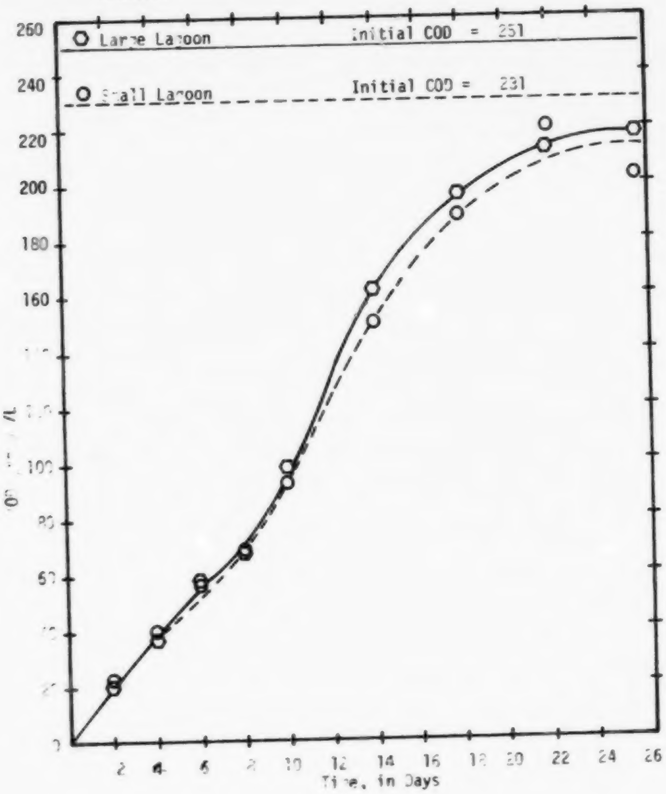

Table 1 – Summary of Oxygen Demand of Lagoon Effluent Based on the 26-Day BOD Determinations.

		Dissolved		Suspended	
Parameter	Total Effluent	Value	% of Total	Value	% of Total
I. Initial COD, (mg O_2/L)	228	91	32	197	68
II. Initial Volatile Solids, (mg/L)	282	126	45	156	55
III. BOD-Values (mg O_2/L)					
5-day	55	8	14	47	86
10-day	94	14	15	80	85
14-day	120	16	13	104	87
26-day	290	44	15	246	85
IV. 26-day BOD Expressed as % of Initial COD	100%	48%		125%	
V. COD Remaining on the 26th Day (mg O_2/L)	142	77	54	65	46
VI. % COD Remaining on the 26th Day	50%	85%		33%	

These data indicate that BOD_5 of lagoon effluent includes only about 20 percent of the oxygen demand exerted by these materials in a 26 day period. Much of the oxygen demand of these effluents appeared to be associated with nitrification, which the COD method used in this study does not include (2). However, in one case, with two effluents dominated by different organisms, the 26 day BOD was equivalent to 90 percent of the measured COD and in another case it was equal to 100 percent of the measured COD in the total effluent. Considering both long-term BOD studies, it appears that COD is a much better estimate of oxygen demand of lagoon effluents than is BOD_5, and that about 95 percent of the measured COD will be exerted as oxygen demand in a 26 day period.

It also should be noted from Table 1 that the dissolved fraction of these effluents is relatively non-biodegradable and that most of the oxygen demand associated with lagoon effluents is due to the suspended fraction comprised of living organisms. In fact, oxygen demand equal to 125 percent of the COD of the particulate portion is exerted in 26 days with an additional 33 percent remaining as humus. Obviously, even COD is not a particularly accurate measure of the oxygen demand of the living organisms found in lagoon effluent; but when applied to the entire effluent it appears to yield a reasonable estimate of "ultimate" oxygen demand.

During the early portion of this study the effluent was drawn from a depth of 18 inches. However, by early August the lagoons had accumulated nuisance amounts of algae and the level of withdrawal was raised to a depth of eight inches on August 9, 1967. Following this change in operation the longitudinal profiles for stream water COD became quite eratic below the outfalls.

A study of the diurnal variation of the effluent of the large lagoon was conducted to see if the effluent remained relatively constant over a 24 hour period. Measurements of effluent discharge and COD of both total and filtered effluent were obtained at two hour intervals for a period of 28 hours. The results of the investigation, shown in Figure 3, indicate that while there was a considerable diurnal variation in the COD of the particulate fraction, the dissolved organics remained essentially constant. This variation in particulate COD was associated with diurnal vertical movement of the algae within the lagoon. From Figure 3 it can be seen that such diurnal algal migration can alter the amount of organic matter discharged from a lagoon by at lease a factor of 4. The possibility of such diurnal variability obviously should be considered whenever lagoon effluents are monitored.

Estimates of the average daily and total organic load discharged by the two lagoons to Bear Creek from July 1 to October 1, 1967, are given in Table 2. Based on the long-term BOD studies the total oxygen demand added to the stream during this three month period was in excess of 33,000 pounds of oxygen while the total addition of caloric energy to the stream from the two lagoon effluents exceeded 51 million kilogram calories. These high values would seem to indicate that these effluents added a significant amount of organic material which would be subject to further degradation in the stream.

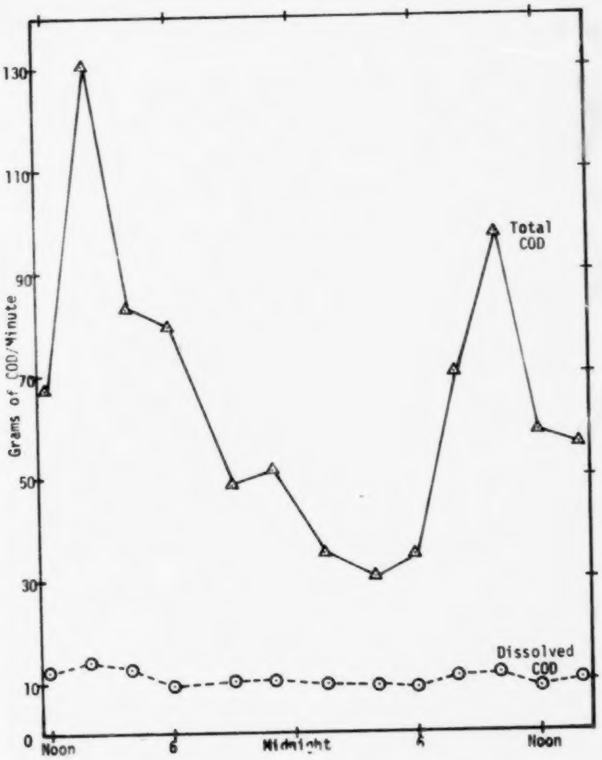

Figure 3. Diurnal variation in effluent from the large lagoon.

Table 2 – Average Daily Loading of Bear Creek from the Two Municipal Lagoons Given with the Estimated Total Load from July 1, 1967 to October 1, 1967.

COD	Dissolved	Suspended
Small Lag.	39.3 lbs (17.8 kg)0_2/day	84.2 lbs (38.1 kg)0_2/day
Large Lag.	66.9 (30.3 kg)	170.5 (77.3 kg)
Combined	106.2 lbs (48.1 kg) 0_2/day	254.7 lbs (115.4 kg)0_2/day
7/1 - 10/1	9,770 lbs (4,425 kg)	23,400 lbs (10,610 kg)
26-Day Oxygen Demand		
Small Lag.	>18.8 lbs (8.5 kg)0_2/day	>103.0 lbs (46.7 kg)0_2/day
Large Lag.	>32.1 (14.5 kg)	>208.0 (94.2 kg)
Combined	>50.9 lbs(23.0 kg)0_2/day	>311.0 lbs(140.9 kg)0_2/day
7/1 - 10/1	>4,680 lbs (2,115 kg) 0_2	>28,600 lb (12,910 kg) 0_2
Energy		
Small Lag.	39,400 kcal/day	156,000 kcal/day
Large Lag.	65,240	298,670
Combined	104,640 kcal/day	454,670 kcal/day
7/1 - 10/1	9,630,000 kcal	41,750,000 kcal

THE FATE OF THE LAGOON EFFLUENT IN THE STREAM

Bear Creek is the typical small stream consisting of a linear series of riffles and pools, and, like other streams, normally contains very little plankton. Thus, the planktonic algae from the lagoon might be expected to settle rapidly to the bottom of the downstream pools. In October of 1966 the large pools 200-400 yards below the lower outfall contained between two and twelve inches of organic ooze which, when disturbed, released large volumes of anaerobic gases. In June 1967, these pools were free of organic ooze and several Bluegill sunfish were observed spawning in the gravel at the upper and lower ends of the pools. However, by September 1967, these pools once more were blanketed with decaying organics. It appeared that the pools below the lagoon served as sedimentation and storage basins for lagoon algae during dry summer periods and the resulting organic ooze was flushed out by the autumn rains.

To test this hypothesis a complete profile of COD was obtained for the portion of the stream from above the small lagoon to a point 1900 yards below the large lagoon outfall. Included were COD measurements of stream water, stream water filtered through a 0.45 μ membrane filter, and sediment collected in petri dishes placed on the stream bottom.

Average values for COD of both filtered and unfiltered stream water at the various sampling stations are plotted in Figure 4 and are given with their respective 95 percent confidence coefficients in Table 3. Two hundred yards below the second outfall the confidence interval for unfiltered stream water COD was quite wide at ±38.7 mg COD/1 (±60 percent of the mean) but it progressively narrowed to ±9.6 mg COD/1 (±30 percent of the mean) 1900 yards below this outfall. This downstream difference in statistical variation is attributed to the variation in effluent COD associated with diurnal vertical migration of algae after the drawoff level was raised in early August. The decreasing variation in COD of unfiltered stream water was due to settling of particulate matter in the pools. The bulk of this sedimentation occurred above the 1100 yard station. Much of the variation in stream water COD below the lagoons is attributed to the variation in algal standing crop exported from the lagoons throughout the summer. The narrower confidence intervals associated with the COD of the dissolved material again indicate its relatively constant addition from the lagoons throughout the summer.

Considering the data collected during the entire summer the effect of the two lagoons was to increase the average total COD of the stream from 6.9 mg 0_2/1 at the park immediately above the small lagoon to 31.7 mg 0_2/1 at a point more than a mile below the second outfall. The corresponding COD values for the dissolved material at these two stations were 6.4 and 19.1 mg 0_2/1 respectively.

In general the green algae and the euglenoids were deposited readily in the pools in the upper portion of the study section. Some of the smaller bluegreen algae, buoyed up by their pseudovacuoles, floated much further. In the lower portion of the study section most of the bluegreen algae which were removed were stranded on the sand bars and in the quiet water near shore.

Figure 4. Average COD for stream water throughout the Bear Creek study section.

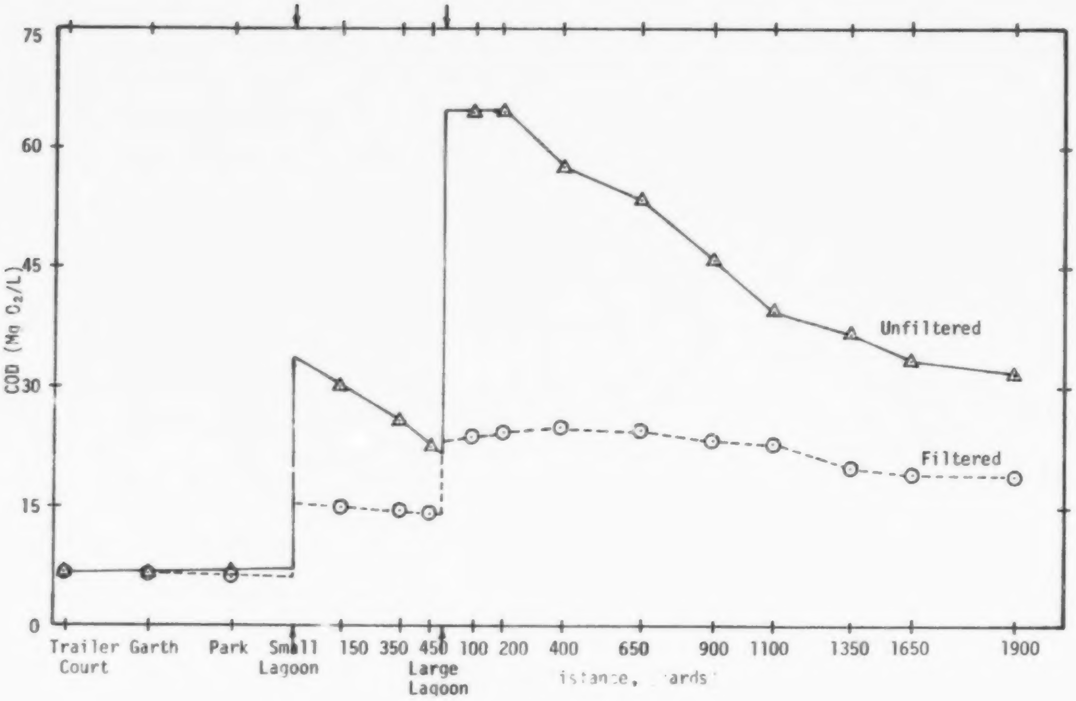

Table 3 – Average COD Values for Filtered and Unfiltered Stream Water at each Sampling Station in Bear Creek Given with the 95 Percent Confidence for the Means.

STATION	UNFILTERED AVERAGE COD (mg O_2/L)	95% C.C.	FILTERED AVERAGE COD (mg O_2/L)	95% C.C.
Park	6.9	1.3	6.4	4.0
Small Lagoon Outfall				
150 Yards	30.0	14.0	15.0	6.1
350 Yards	25.5	6.5	14.7	5.6
450 Yards	22.4	7.7	14.2	2.3
Large Lagoon Outfall				
100 Yards	64.6	27.1	23.9	3.0
200 Yards	64.7	38.7	24.2	4.5
400 Yards	57.6	33.1	25.0	4.4
650 Yards	53.6	31.4	24.4	3.1
900 Yards	46.0	23.9	23.3	3.0
1100 Yards	39.6	16.1	22.8	2.4
1350 Yards	36.7	15.0	19.8	2.9
1650 Yards	33.3	11.9	19.3	2.2
1900 Yards	31.7	9.6	19.1	2.4

Based on downstream changes in stream water COD there appeared to be two different zones of sedimentation in the study section. In the first zone, extending downstream 1100 yards below the lower outfall, the average rate of sedimentation, corrected for groundwater infiltration was 9.0 grams of COD/M^2/day. The average settling rate over the remaining 800 yards of the study section was 1.7 grams COD/M^2/day. The increase noted in the COD of the dissolved material in the upper 1100 yard zone was attributed to lysing of algal cells after they had settled to the bottom of the pools. The readily biodegradable fraction of this material was utilized by the heterotrophic organisms in the riffles and appeared to be largely removed by the time the stream reached the 1350 yard station.

There appeared to be a significant change in the stream at about the 1100 yard station. Above this point the riffles largely were covered with *Sphaerotilus* while below this sampling station the attached growth in the riffles contained increasing amounts of periphytic algae. These profuse growths on the riffles, resulting from biological uptake of organics and photosynthesis, undoubtedly added to the COD of the stream when they were scoured from their attachment surface.

An attempt was made to separate the fraction of the stream water COD due to lagoon algae from that due to periphytic growths scrubbed from the stream bottom. The model used to separate these components was based on COD measurements of stream water, filtered stream water, and sediments collected over 24 hour periods in petri dishes placed on the stream bottom. The basic assumption was that sedimentation within the petri dishes was representative of that which occurred on the stream bed.

The model was based on the assumption that the net accrual of organic material on the stream bed was the combined result of sedimentation of lagoon solids, sedimentation of periphytic organisms scrubbed from upstream, growth on site, lysing of deposited solids, and some utilization of the dissolved organics. Expressed algebraically it took the following form:

Net accrual = Sedimentation - (lysing-utilization)
+ growth + bed load transfer of
periphytic growth from upstream

On a COD basis net accrual was the COD of the sediment collected in the petri dishes. Sedimentation was calculated as change in the COD of suspended materials between upstream and downstream sampling stations. The quantity (lysing-utilization) was calculated as the change in COD of the dissolved material between sampling stations. Studies of the colonization rate of the periphyton indicated that growth of periphytic organisms on clean substrates was slow enough that it could be ignored over a 24 hour period. The remaining term, bedload transfer, is a measure of periphytic growth, including some enmeshed lagoon solids, scrubbed from the riffles immediately above the pools in which the petri dishes were placed. This parameter should be a good index to the growth rate of periphytic organisms throughout the study section.

Assuming growth over a 24 hour period to be negligible and rearranging terms yields the following model for calculating bedload transfer or bioactivity.

$$COD_{bioactivity} = COD_{accrual} - \Delta COD_{suspended} + \Delta COD_{dissolved}$$

The results of this calculation for the entire study section are given in Figure 5 where it appears that there are two separate zones of bioactivity with a transition point at about 1100 yards. The low value for bioactivity between the 200 and 400 yard stations is attributed to the two long, deep pools with anaerobic bottoms and the near absence of riffles in this area of the stream which provides very little suitable substrate for periphytic growth. If the histogram in Figure 5 from 400 yards below the upper outfall is transposed to 400 yards below the lower outfall, it would fit the general shape of the bioactivity curve. It appears that had the length of stream 200-400 yards below the lower outfall been riffles and small pools, instead of deep pools, the departure from the general shape of the bioactivity curve would not have occurred.

These data indicate that there were two different zones of bioactivity below the lower lagoon outfall. The first zone extended about 1100 yards downstream and was due primarily to production of *Spaerotilus*. The marked downstream decrease in *Spaerotilus* activity probably was due to a concomitant decrease in a suitable food for them. The appearance of many diatoms below 1100 yards suggests that the increased bioactivity in this area of the stream may indicate improved conditions which allow periphytic algae to successfully compete with the heterotrophic organisms and to begin to take advantage of the primary nutrients added to the stream by the lagoon. This second zone of activity began at about the same point where the settling rate of COD due to suspended matter decreased markedly. This suggests that the rate of oxygen utilization by the decomposing sediments may have resulted in dissolved oxygen levels which limited all but the more tolerant forms above the 1100 yard point.

Bottom fauna data collected in November, 1966, from riffles throughout the study section also indicate a rather abrupt change at about the 1100 yard station as shown in Table 4. The low diversity and standing crop of macrofauna above the lagoons probably was due to the chromate used at the municipal power plant; the effluent from which comprised most of the dry weather flow of the stream. The increased diversity at about 1100 yards is in agreement with both the bioactivity index and the downstream decrease in stream water COD. The difference in types of organisms found above and below the 1100 yard station again suggests that the rather sharp demarcation at this point may be associated with oxygen availability.

The changes in the biota in Bear Creek at the 1100 yard point appear to be similar to those noted by Gaufin and Tarzwell (3) in Lytle Creek three miles below the outfall of primary treated sewage. However, Lytle Creek was 90 percent sewage effluent during low-flow conditions while only 25 percent of Bear Creek was lagoon effluent during dry weather flow. If the length of stream affected is a direct function of the degree of dilution, it would appear that at a waste dilution in Bear Creek similar to that in Lytle Creek the change noted at 1100 yards would have occurred 2.25 miles below the lagoon outfall. This suggests that while single cell lagoons yield a coliform bacteria

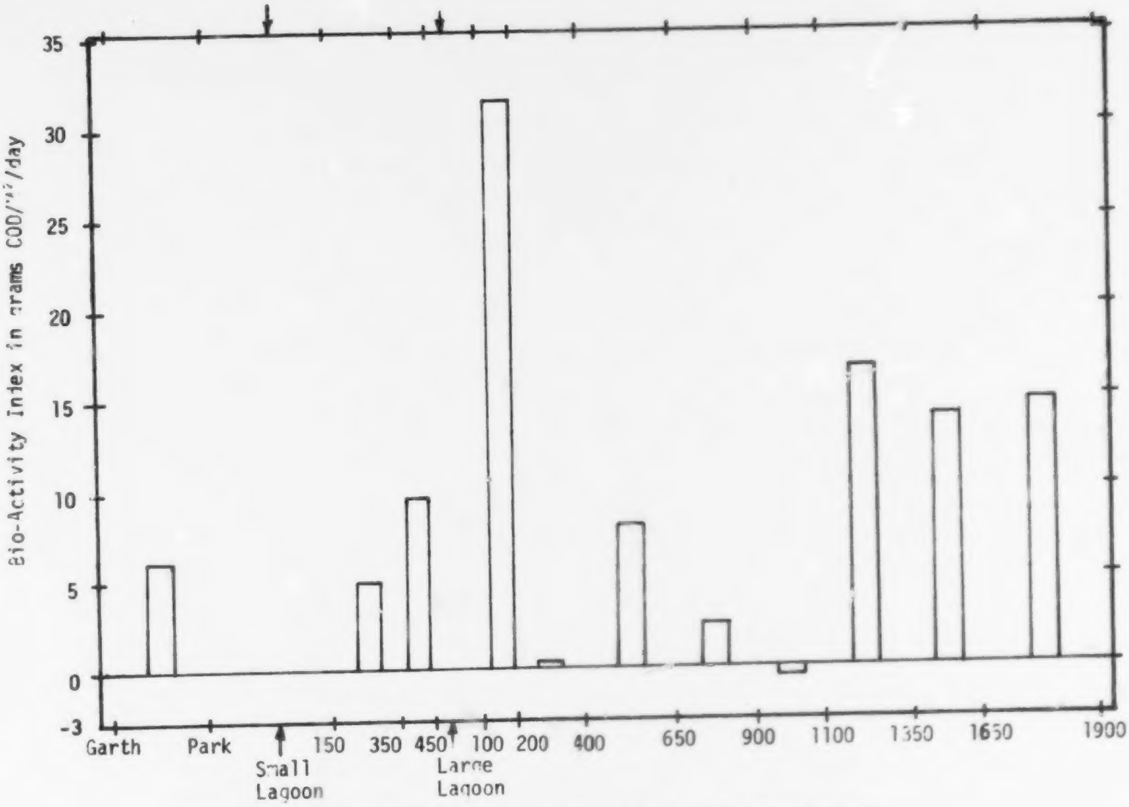

Figure 5. Bioactivity index for periphytic growth throughout the Bear Creek study section.

Table 4 – Macrofauna Data Collected from Selected Riffles In Bear Creek During November 1966. Units are Numbers per Square Foot.

			Small Lag.	Large Lag.			
Location:	Garth Ave.	Park	150 Yds	550	1000	1200	1650
Tubificids		1	14,000	10,000	2,000	Numerous	Numerous
Other Oligochaetes	2						1
Chironomids	16	67	224	131	185	335	18
Hydropsychidae			2		23	120	4
Ephemeroptera	2				2	4	1
Similiidae					3	38	30
Tabanidae							2
Hymenoptera						1	
Elmidae							20
Other Coleoptra					1		
Asellus						1	
Leeches						2	3
Snails		5	1				
Mussels						20	1
No. of Groups/ft^2	3	3	4	2	6	9	10
Standing crop Ash-free dry weight (mg/ft^2)	1.5	36.7	228.9	161.7	298.0	960.5	399.8

reduction equal to secondary treatment, the organic removal during the summer more nearly approximates that of primary treatment.

It also should be noted that during the summer of 1967 accrual of lagoon algae on the stream bottom resulted in significant amounts of anaerobic sludges. Such accumulation of reduced material has significant potential for deoxygenating water. The first heavy rain following prolonged dry weather may scour these sediments causing significant deoxygenation of the stream water.

This study did not include a detailed investigation of dissolved oxygen throughout the study section so little can be said about actual dissolved oxygen concentration in the stream. However, the average rate of deposition of COD in that section of the stream from 200 to 400 yards below the lower outfall was equal to a 26-day BOD of 675 mg $0_2/M^2$/hour. At the average summer temperature of about 20°C. and assuming oxygen saturation at 200 yards and no physical reaeration or photosynthesis, the average daily flow through this section of stream would supply 626 mg $0_2/M^2$/hour. Thus, it appears that there was a significant deoxygenating potential added to just this 200 yard portion of the stream.

The data collected from this study indicate that the algae in lagoon effluents are a definite detriment to the receiving stream. However, the absence of dissolved oxygen data during this study did not allow any evaluation of the potential for photosynthetic reoxygenation by lagoon algae in the stream.

A study was undertaken in the summer of 1968 to evaluate the daily oxygen budget in the large pool just above the 200 yard station below the lower outfall. Unfortunately scouring rains occurred at least every fourth day throughout the summer of 1968 and there was no significant accumulation of organic debris in the pool at any time during that summer. Thus, the oxygen budget obtained reflects the best conditions in the stream and cannot be applied directly to the dry weather conditions of 1967.

Measurements of dissolved oxygen concentration were obtained at hourly intervals for 24 hour periods at each of five sampling stations in the 287 foot long study section. The upper 17.6 foot long portion of this section is a riffle and the remainder is a large pool with a maximum depth of about 4.5 feet. Reaeration coefficients were calculated for each sampling station and photosynthetic oxygen production and total respiration were calculated from diurnal changes in dissolved oxygen as the water flowed through the study section. Average values, based on entire 24 hour periods, were obtained for total respiration, total reaeration and net and gross photosynthetic oxygen production. In the 3.5 hour period it took the average liter of water to pass through the 287 foot study section, an average total respiration of 3.027 mg $0_2/1$ was exerted. The average value for gross oxygen production, which includes all of the oxygen produced by algal photosynthesis in the study area, was 2.523 mg $0_2/1$. The average reaeration was 0.709 mg $0_2/1$. Thus, the average liter of water passing through this riffle and pool lost 3.027 mg oxygen as respiration and gained 2.523 mg oxygen from algal photosynthesis and 0.709 mg oxygen from reaeration for a net gain of about 0.2 mg oxygen.

These data indicate that while there was considerable photosynthesis by lagoon algae in this portion of the stream, it did not meet the respiratory demand. In the absence of reaeration, most of which was accomplished in the riffle above the pool, the average dissolved oxygen concentration would have decreased by about 0.5 mg $0_2/1$ through the study section. This area of the stream contains more planktonic lagoon algae than any of the downstream reach and photosynthesis should be at a maximum in this area. During the period of this study there was no accumulation of organic debris on the pool bottom so the above data reflect the best conditions in this portion of the stream. Significantly greater respiration rates would be expected with the large anaerobic sludge accumulation characteristic of the dry weather flow of 1967.

It should be noted that photosynthetic oxygen is not free oxygen and that new algal protoplasm is formed as this oxygen is released. Oxygen supplied by algal photosynthesis in the stream can be used to support heterotrophic respiration; but when such oxygen is used for this purpose the stream incurs an oxygen debt which must be paid off when these algae settle to the bottom of downstream pools.

CONCLUSION

Algae-laden lagoon effluents can influence markedly the environment of a receiving stream for a distance of many yards to several miles downstream. The length of stream affected in each case depends on the degree of dilution of the waste effluent and on the riffle-pool ratio of the receiving stream. Small streams receiving lagoon effluent appear to be little more than a linear series of trickling filters (riffles) and sedimentation and digestion units (pools). As such, a stream receiving lagoon effluent is an integral portion of the total waste treatment system and this utilization of a stream should be recognized and acknowledged whenever lagoons are used.

ACKNOWLEDGEMENTS

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EFFECTS OF AN OXIDATION POND EFFLUENT ON RECEIVING WATER IN THE SAN JOAQUIN RIVER ESTUARY

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James A. Robertson and William H. Pierce[*]

INTRODUCTION

The San Joaquin River, during the summer months, is laden with phytoplankton as a result of nutrient enrichment by agricultural and municipal waste discharges. Oxidation pond effluent from the City of Stockton, one of the major food processing centers in California, is discharged into the San Joaquin River at a point along the southeastern edge of the Sacramento-San Joaquin River Delta. Reduced fresh water outflows in the San Joaquin River and the heavy seasonal quantities of wastes from oxidation ponds of the City of Stockton have been accompanied by frequent low oxygen levels approaching asphyxial levels with resultant fish kills in a reach of the deepened ship channel of the San Joaquin River below Stockton. The San Joaquin River has, until recently, been a major migration route for King Salmon returning from the Pacific Ocean to spawning beds in tributary streams of the San Joaquin River. Reduced king salmon runs, in recent years, may be due in part to depressed dissolved oxygen levels during the fall in the San Joaquin River downstream from Stockton.

The purpose of studies described herein are to identify causes of the dissolved oxygen depressions in the San Joaquin River downstream from Stockton, California. Field studies to provide data for this analysis were conducted during the period of July to October, 1964, August and September, 1967, and September and October, 1969. Reports covering the 1964 and 1969 surveys have been published. (1) (2).

The scope of the study was limited to an analysis of factors considered related to the dissolved oxygen problem. The factors included water temperature, algae, oxygen demands, nutrients, flow and channel characteristics.

The study area is located along the southeastern border of the Sacramento-San Joaquin River Delta in San Joaquin County (See Figure 1). The study was confined to a 23 mile reach of the San Joaquin River between Turner Cut (Station 1) on the north and the Mossdale Bridge (Station 7) on the south. The City of Stockton, (population 125,000) with several food processing plants and other industrial activities, is the principle community in the study area.

Seven sampling stations were selected and are shown on the study area map, Figure 1. Four of these stations (1-4) are in the 30 feet deep Stockton ship channel; the remaining three (5-7) are in the shallow San Joaquin River to the south. Samples were also collected from the Stockton Sewage Treatment oxidation pond effluent structure, which is located near Station 5. Table 1 gives a description of the sampling stations.

The entire stream reach studied is under tidal influence but contains no measurable quantities of ocean salts. The mean tidal range, at the Mossdale Bridge, is approximately 2.4 feet; at Turner Cut, approximately 4.0 feet. From Mossdale Bridge to Stockton, a distance of approximately 16 miles, the stream is relatively shallow, ranging from 5 to 15 feet in depth at mean tide. Stream width varies from approximately 150 to 300 feet. The channel in the remaining 9 miles from Stockton to Turner Cut, has been broadened and deepened to accommodate shipping. The channel depth averages approximately 30 feet at mean half tide and is about 600 feet wide.

The junction of the shallow reach and the deep ship channel has been broadened and deepened to form a settling basin for the removal of sediments. From the settling basin eastward, the deep ship channel extends to the Port of Stockton where the channel is further broadened to form a turning basin for ocean going freighters. The movement of water in this deadend reach is mainly due to tidal action. The first and most intensive study period extended from July 13 through October 27, 1964. During early July, the oxidation pond was not discharging; this provided an opportunity to sample the river when the entering waste canal water releases from the Delta Mendota Canal were provided by the U.S. Bureau of Reclamation (USBR) at the request of the California Department of Fish and Game to assist migration of king salmon. A partial barrier of Old River (See Figure 1) and deliberate release of Delta Mendota Canal water into the San Joaquin River south of the sampling points provided significantly greater flushing of the San Joaquin River channel.

During the initial surveys in 1964 eight cruises were scheduled, each consisting of a late afternoon and pre-dawn sampling run. The dates are listed in Table 2. Sampling was initiated at Station 1 near Turner Cut and proceeded upstream to Station 7 at Mossdale Bridge. The time required to complete a run was two-three hours, depending on river conditions and the number of samples taken.

Surveys in the Stockton area during the late summer of 1967 included measurements of diurnal changes in oxygen concentrations, oxygen production and demand using dark and light bottle technique and measurement of *in situ* respiration in benthic sediments. These efforts were

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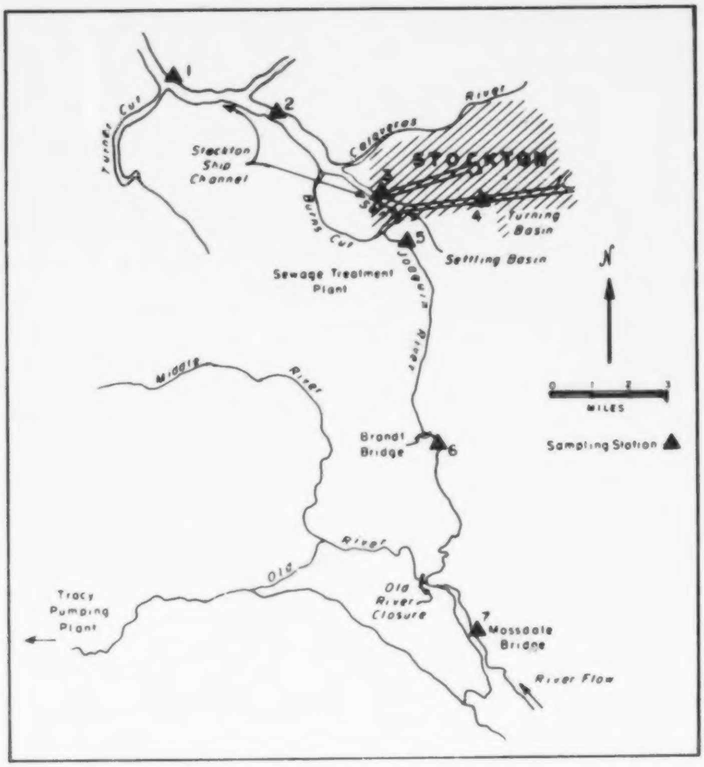
1
2
Turner Cut
Stockton Ship Channel
Calaveras River
STOCKTON
3
4
Turning Basin
Burns Cut
5
Settling Basin
Sewage Treatment Plant
Joaquin River
N
0 1 2 3
MILES
Sampling Station ▲
Middle River
Brandt Bridge
6
Old River
Tracy Pumping Plant
Old River Closure
7
Mossdale Bridge
River Flow

Figure 1. Study area, San Joaquin River Estuary near Stockton, California

Table 1 – Sampling Station

Station	Location Description	River Miles Above Station 1
1	Ship Channel near Turner Cut, light 23	0.0
2	Ship Channel between lights 35 and 36	3.0
3	Ship Channel near light 45	5.8
4	Turning Basin	*
5	River, under Santa Fe R.R. Bridge	8.7
STP	Stockton Oxidation Pond Discharge	8.0
6	River, Brandt Bridge	14.4
7	River, Mossdale Bridge	22.7

* Not in hydraulic sequence

Table 2 – 1964 Cruise Schedule

Cruise	Dates
1	July 13 - 14
2	July 30 - 31
3	Aug. 12 - 13
4	Aug. 27 - 28
5	Sept. 10 - 11
6	Sept. 26 - 27
7	Oct. 10 - 11
8	Oct. 26 - 27

coincident with an intensive flow measurement program by the U.S. Army Corps of Engineers.

A limited treatment plant survey was conducted from September 30 to October 2, 1969, to evaluate the efficiency of plant operation and maintenance, in accord with agreements with the city under the PL 660 construction grant program to construct, operate, and maintain waste treatment facilities to assure compliance with water quality standards adopted for the receiving waters into which treated effluent will be discharged. The survey was a part of an integrated investigation of waste treatment and quality of receiving waters and included a 24 hour evaluation of treatment plant efficiency for removal of carbonaceous materials, nitrogen, phosphorus, and pesticides, and an estimate of the waste loading on the San Joaquin River near Stockton.

All river water samples were collected in Kemmerer sampler on the same slack tide in order to minimize variation due to tidal excusion. In the study area water movement due to tide is pronounced and sampling without reference to tide would introduce large inconsistencies in the data. A low slack tide was sampled in the late afternoon and early the following morning in the 1964 and 1969 surveys. With the effect of tide minimized, changes in dissolved oxygen could be related to other causes, such as respiratory and photosynthetic activity related to algae and other biota. Samples during diurnal studies in 1967 were collected every three hours.

Samples of raw sewage, secondary plant effluent and pond effluent were collected between September 30 and October 1, 1969. Raw sewage and secondary effluent composite samples collected during the 1969 survey were prepared from one hour grab samples. Four composite samples of pond effluent representing six hour increments were collected by means of a battery operated continuous automatic sampler.

All samples were either iced at the point of collection or refrigerated at the plant laboratory, following collection. To prevent contamination or loss by absorption, an aluminum foil cover was inserted over the mouth of each sample bottle, prior to capping.

The analyses, conducted on each of the composite samples included:

(a) NH_3-Nitrogen
NO_3-Nitrogen
Organic Nitrogen

(b) Ortho phosphorus
Total phosphorus

(c) Organic carbon
Carbonate alkalinity
Bicarbonate alkalinity

(d) B.O.D. (5 day)
B.O.D. (30 day)
C.O.D.

Water quality determinations were generally based on Standard Methods. (3) Exceptions included use of special equipment used such as a benthic respirometer patterned after one designed by O'Connell and Weeks, (4) and a fluorometer for *in vivo* chlorophyll measurements based on the work of Lorenzen, (5) Phytoplankton samples collected from the oxidation pond and the river were preserved in neutral formalin and counted in a Sedgwick-Rafter cell. Chlorophyll extraction of discrete samples from the river and ponds were by the fluorescence technique described in Yentsch and Menzel. (6).

The City of Stockton is one of the major food processing centers in California. The processing of asparagus and spinach in April marks the start of the cannery season, which usually extends through December when prunes and pumpkins are processed. The heaviest production period occurs in August, September and October with the processing of peaches and tomatoes. Other fruits and vegetables, which are handled in Stockton canneries include apricots, cherries, and peas. The City of Stockton provided waste treatment for six food processing companies in 1964. By 1969 the city treated food processing wastes from eight canneries and one meat packing plant and a paper products industry.

Waste treatment facilities at Stockton consist of secondary treatment by trickling filters and retention in two oxidation ponds. The two oxidation ponds occupied approximately 220 acres in 1964 and were expanded to 325 acres by 1967 and 480 acres by the fall of 1969.

Waste loading to the river from the oxidation ponds may be typified as an algal culture generally composed of a mixed population of coccoid green algae often dominated by the genus, *Scenedesmus*. Waste flows varied from monthly averages of 10 - 25 MGD in 1964 to 32 MGD during September, 1969. The San Joaquin River also brings oxidizable materials including algae to the Stockton channel area. River flows during late summer range from around 700 cfs net upstream to about 1500 cfs net downstream flow. Upstream flows occur when demands at the U.S. Bureau of Reclamation Tracy Pumping Plant create net upstream diversion of water through Old River. Figure 2 illustrates the estimated time of travel through the study area at various flows.

During the 1964 survey a partial closure of Old River was effected to augment the flow past Stockton. Flow was increased further after September 23 following release of water diverted at the Tracy Pumping Plant and released into the San Joaquin River via the Delta Mendota Canal. The flow past Stockton during the 1964 study period, Figure 3 depicts the estimated net flow past Stockton observed during the 1964 survey.

During the September 1967 survey the Corps of Engineers measured velocities in the study area and found maximum mid-depth velocities of 1.1 fps at flood and 1.5 fps at ebb tide in the shallow river near station 5 and 0.8 fps and 0.9 fps at flood and ebb respectively at station 2 in the deep water channel. No closure was in effect; estimated flow past Stockton during this survey was 400 cfs.

During the 1969 survey flow past Stockton was estimated to be 1306 cfs; no Old River closure was in effect.

1964 SURVEY RESULTS

Information collected during the course of studies in 1964 are described in three succeeding sections on physical, chemical, and biological data.

Physical Data

Water temperatures were essentially constant with respect to depth in both the shallow well-mixed river and in the deep water ship channel. The range of water temperatures was approximately 24° - 26°C in July and August and dropped to 16° - 18°C by late October. Water temperatures observed during the morning cruises were generally 1 - 2°C lower than in the afternoon cruise. Station 1 was coolest in July and warmest in October after river

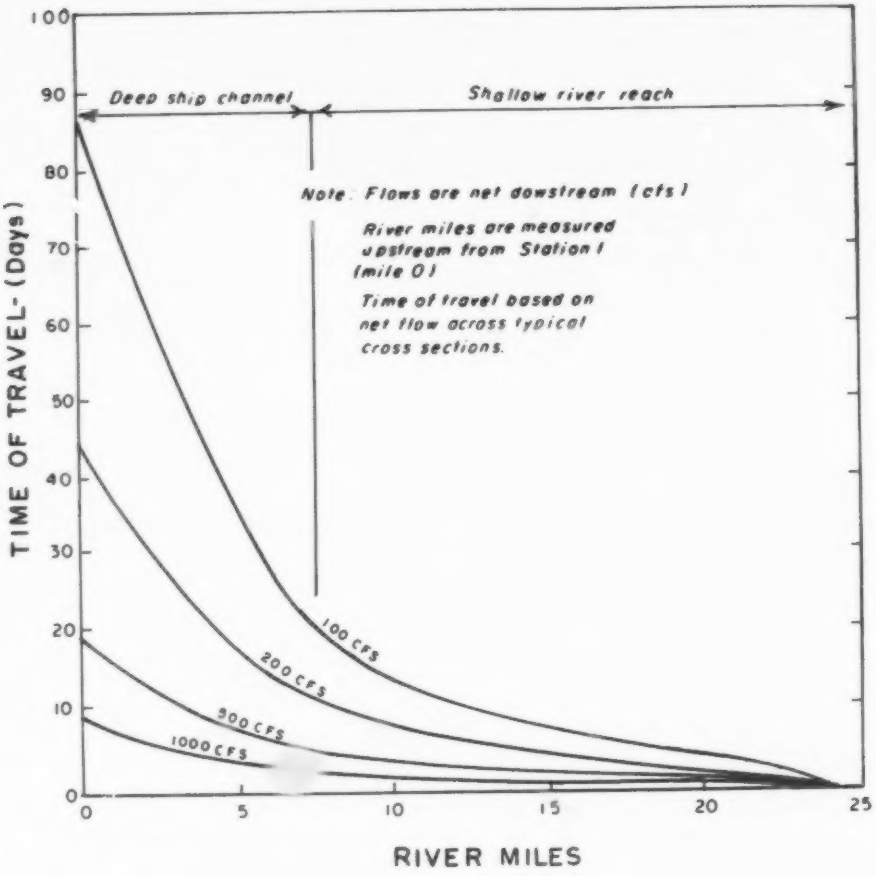

Figure 2. Time of travel based on net downstream flows through the study area.

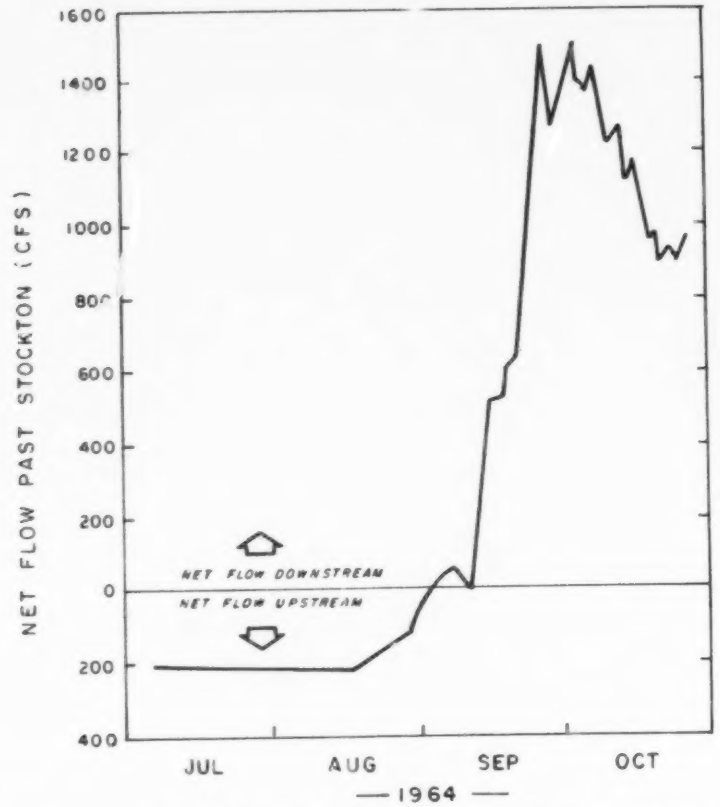

Figure 3. Estimated net flow past Stockton during the 1964 study period.

water temperatures had dropped.

Light penetration in the study area is quite limited. No organized light penetration study was conducted, but visual observations and field tests made since 1964 indicate that transparency values (Secchi disc) are often about one foot in the shallow river and one to three feet in the ship channel during the summer and early fall. Secchi disc depths reach to one half to one third the euphotic zone.

Chemical Data

A summary of total nitrogen (N) and total phosphorus (P) from the samples (where all forms were reported) is given in Table 3. In the early cruises (1-4) when net flow was upstream, the ship channel stations had lower concentration of nutrients than occurred after flow was augmented. Quite the opposite relationship for phosphorus appears evident in the river stations. Total nitrogen appears more constant in the shallow river.

Dissolved oxygen (DO) measurements were made on the same tidal slack late in the afternoon and early in the morning on each of the eight cruises. Measurements were also taken at several depths in the ship channel and at deep sections in the river. The results can thus be compared according to spatial variations (with water depth and from one station to another) and temporal variations (between the afternoon and morning cruises, or between the bimonthly sampling runs.)

The DO concentrations measured during the study are presented in Table 4. The afternoon and morning DO concentrations for near surface samples at all stations for cruises 1-4 are plotted in Figure 4.

Profiles (Figure 4) of the average afternoon and morning DO levels reflect the different character of the shallow river reach and the ship channel section, as well as the photosynthetic activity of algae. The higher DO (both top and bottom, Table 4) in the shallow river section, as compared with the deep ship channel is clearly evident.

Dissolved oxygen levels in the ship channel during cruises 1-5 as compared with cruises 6-8 (Table 4) demonstrate the effect of increased river flow from late September through October. For example, at Station 3 during cruises 1-4, the DO of all ten bottom samples (5 morning and 5 afternoon) was equal to or less than 5.0 mg/1; whereas, during cruises 6-8 the DO of the six bottom samples was always greater than 6.0 mg/1. The poor quality waters of Station 3 appears to have been displaced downstream by the flow release since DO levels dropped at Station 1 during cruises 6-8. The average bottom DO at Station 1 during cruises 1-4 was 6.2 mg/1. The poor quality waters of Station 3 appears to have been displaced downstream by the flow release since DO levels dropped at Station 1 during cruises 6-8. The average bottom DO at Station 1 during cruises 1-4 was 6.2 mg/1 as contrasted with 5.3 mg/1 during cruises 6-8.

In the river reach the DO differences (from afternoon to morning) were less marked during cruises 7 and 8, as compared with the early cruises. This difference may be a result of dilution of planktonic algae and the presence of lower water temperatures in October, which would limit the activity of the algae present.

Table 3 – Nutrients – San Joaquin River Stations

Station Number	Location	Total Nitrogen[a] Cruises (1-4)		Total Phosphorus[a] Cruises (1-4)	
		Cruises (1-4)	Cruises (6-7)[b]	Cruises (1-4)	Cruises (6-8)
1	Ship Channel	1.3	2.4	0.12	0.18
2	Ship Channel	1.5	3.2	0.22	0.25
3	Ship Channel	1.7	3.5	0.23	0.32
4	Turning Basin	1.9	3.0	0.16	0.21
5	River	3.0	2.6	0.44	0.25
6	River	2.4	2.8	0.32	0.17
7	River	2.2	2.4	0.25	0.16

(a) Values listed are in milligrams per liter.
(b) No organic nitrogen data for Cruise 8.

Table 4 – Observed Dissolved Oxygen Concentrations 1964*

	Cruise															
Station	1		2		3		4		5		6		7		8	
	PM	AM	PM	AM	PM	AM	PM	AM	PM	AM	PM	AM	PM	AM	PM	AM
1	7.0 6.0	6.0 5.8	5.8 5.6	5.6 5.4	7.0 6.1	7.2 7.4	6.8 6.8	6.4 6.2	8.0 6.8	7.2 7.4	6.8 5.6	6.0 5.0	5.6 5.0	5.0 5.0	5.5 5.4	6.2 5.8
2	6.0 –	5.4 5.8	4.2 3.8	3.8 4.2	6.2 4.7	5.0 5.4	6.0 4.6	5.0 5.2	8.1 4.2	4.6 4.2	6.2 4.8	5.4 5.0	– 5.2	5.6 5.6	8.4 6.0	7.0 6.6
3	7.0 4.0	4.0 4.0	4.0 3.8	2.8 2.6	5.2 4.0	4.3 5.0	7.4 1.8	3.2 3.2	8.2 4.0	4.3 3.5	11.0 8.0	9.2 8.0	11.4 6.2	7.2 6.8	10.1 6.8	7.6 7.6
4	5.6 3.2	3.8 3.7	6.6 4.4	5.8 4.8	7.8 5.2	6.4 6.2	10.2 5.0	4.8 4.6	8.0 2.6	5.4 2.2	5.4 3.0	5.0 4.8	8.4 6.0	4.7 4.4	11.6 7.0	7.0 5.8
5	7.9 –	5.2 –	8.0 –	5.2 –	7.2 –	6.8 –	8.6 –	3.2 –	9.4 –	6.0 –	14.0 –	9.5 –	8.4 –	7.0 –	8.4 –	8.2 –
6	13.0 –	9.8 –	12.4 12.6	8.8 –	12.4 12.2	8.8 9.0	13.6 13.4	10.1 –	12.4 12.5	9.4 9.6	12.6 12.6	8.4 8.4	8.6 8.6	6.8 6.8	8.3 8.2	7.0 7.4
7	11.8 –	7.4 –	8.0 –	5.6 –	11.2 –	10.0 –	14.0 –	11.4 –	15.1 –	13.0 –	11.2 –	6.8 –	8.2 –	6.7 –	8.0 –	8.0 –

$\frac{\text{*Top}}{\text{Bottom}}$

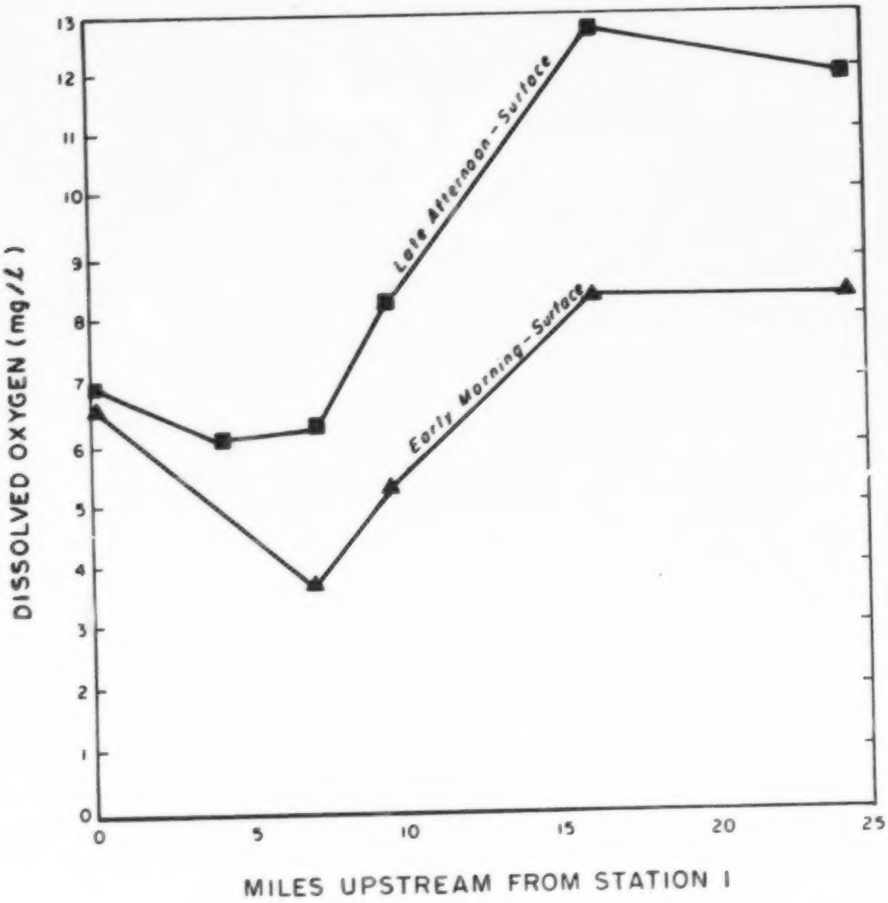

Figure 4. Profiles of dissolved oxygen content of surface waters during 1964 study period (cruises 1-5)

Biological Data

Phytoplankton were identified and enumerated at each station and at all depths sampled on either the morning or the afternoon run of each cruise. In addition, a sample from the oxidation pond discharge was also examined on each but the first cruise when no pond discharge occurred.

The phytoplankton community of the San Joaquin River is to a certain extent homogenous, but differences in the kinds, numbers, and distribution of phytoplankton are apparent from the data. Qualitative differences are most obvious between the oxidation pond and the stations more remote from the pond discharge location. Many of the genera found were not numerous in any samples and do not make up a very significant portion of the biomass; however, the presence of certain genera may have meaning when their environmental requirements are better know. Generally, the dominant phytoplankters in the river and the ship channel are the centric diatoms *Cyclotella* and *Coscinodiscus*. The dominant genera in the oxidation pond are green algae *Chlorococcum*, *Chlorella*, and *Scenedesmus*, and the diatom *Cyclotella*. During the late September and October 1969 when net downstream flow was high, there was a continued increase in phytoplankton at the four channel stations and a decrease at the three shallow river stations. Figure 5 illustrates the pattern of green algal pupulations, the group most indicative of the oxidation pond discharge.

1967 SURVEY RESULTS

The two brief water quality surveys in August and September identified high water temperatures and low oxygen levels in the channel at various water depths from one meter to ten meters. Water temperatures in September were generally near 25°C and oxygen concentrations below 5 mg/1 were found at all deep water channel stations. Figure 6 compares diurnal DO variations at one deep-water channel station and one upstream station.

The 1967 surveys included assessments of oxygen demand of receiving waters and oxidation pond effluents. These data show that river BOD values in the Stockton area are high as compared with values in clean streams. Long term BOD values (25 days) in the Merced River, a relatively

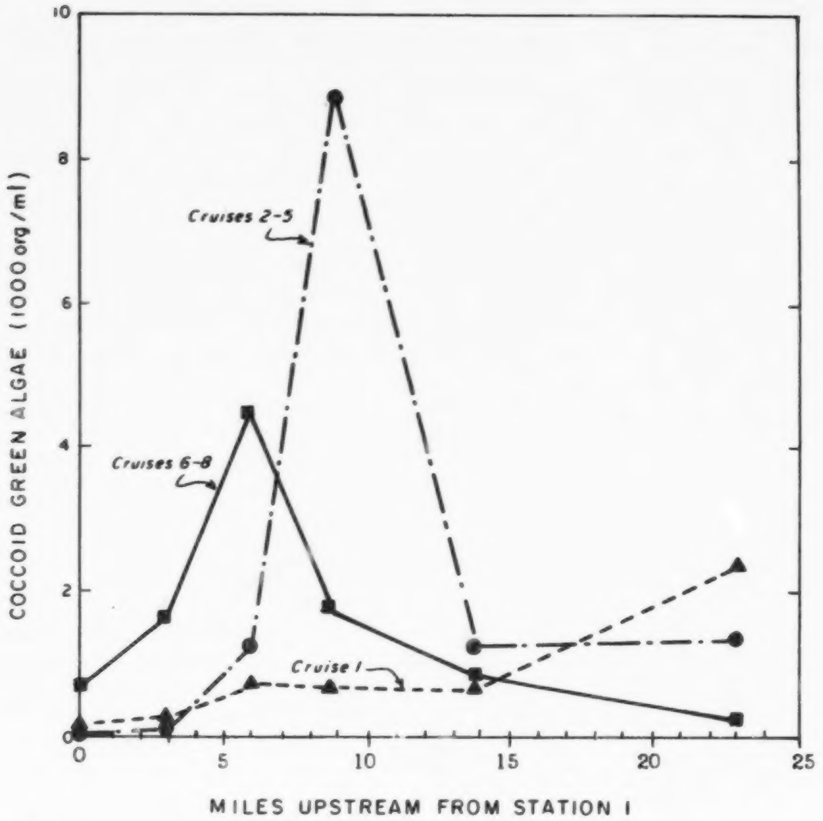

Figure 5. Coccoid green algal populations reflecting oxidation pond discharge at mile 10 and effect of flow augmentation (cruises 6-8)

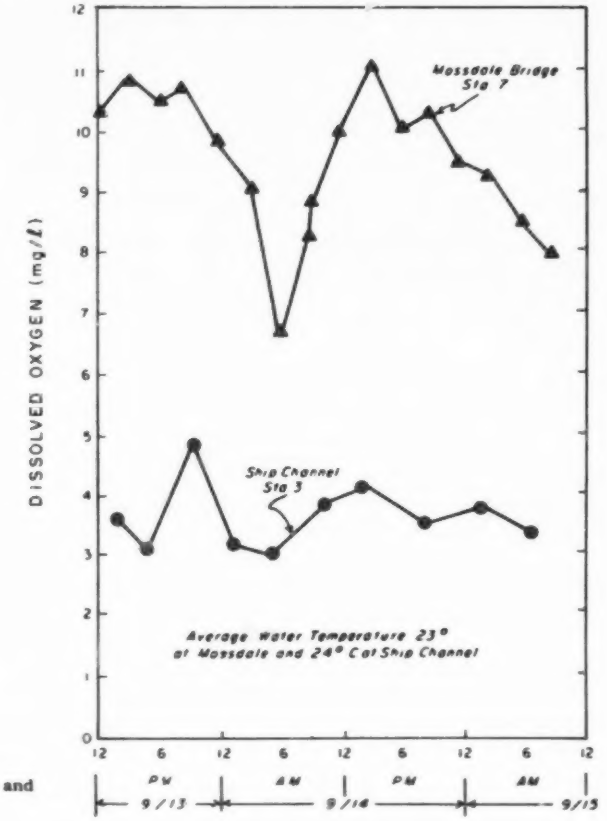

Figure 6. Dissolved oxygen variations typical of shallow river and ship channel during late summer.

clean tributary of San Joaquin River, were 2 - 3 mg/l during the late summer of 1967 in contrast to 23 mg/l between the oxidation pond discharge and the settling basin; in the ship channel downstream from the settling basin long term BOD values fell to 4 - 8 mg/l. Long term oxygen demand (25 days) from Stockton oxidation pond influent and effluent collected during the September survey were 380 mg/l and 220 mg/l respectively. Higher values would have been achieved had the test period been prolonged. Pond effluent samples analyzed by the City of Stockton had a COD of 250-278 mg/l during the same period.

Organic materials including algae settle in the deep-water channel and contribute to oxygen depletions noted in the deeper waters. Direct measurement of the benthic uptake of oxygen was attempted with a respirometer during the September survey, values measured varied from less than 1.0 to 1.6 gms oxygen per square meter per day.

Dark and light bottle measurements were made in the receiving waters and in the laboratory to determine photosynthetic oxygenation rates. Light penetration assumed sufficient for photosynethesis (1% of surface light) was approximately two meters in both the deep channel and shallow river stations. Oxygen production (P) of 5.3 and 8.8 gm/m^2/day was measured at two locations in the ship channel at the same time respiration (R) through the whole water column was assessed near 15.5 and 13.4 gm/m^2/day for a P/R rate of 0.34 and 0.66 respectively. P/R rates less than one typify heterotrophic or oxygen consuming environments. Two shallow upstream areas yielded nearly equal oxygen production rates of 5.9 g/m^2/day with slightly lower respiration through the water column producing a slightly autotrophic or oxygen producing environment.

1969 SURVEY RESULTS

Mean daily flow into the waste treatment plant, during the September 30 - October 1, 1969, study period, was 26.6 mgd. The untreated biochemical oxygen demand loads on the plant were 128,000 #BOD_5/day and 182,000 #BOD_{30}/day. Oxidation pond loadings were 76,000 #BOD_5/day and 102,000 #BOD_{30}/day or 147 and 212 #/acre/day of BOD_5 and BOD_{30}, respectively. Raw sewage, secondary effluent and pond effluent analyses are summarized in Table 5.

Dissolved oxygen demand curves for the raw, secondary and pond effluents are presented in Figure 7. Oxidation of the raw sewage proceeded at a rate of 0.09/day (Base 10). Lags in the oxidation of secondary effluent (presumably due to chlorination) and in the pond effluent were notable. The observed oxidation rate, after acclimation, for the secondary effluent was 0.12/day. Similarly, for the pond effluent, an oxidation rate of 0.08/day was observed.

Waste treatment efficiency, as measured by BOD_5, exceeds 80% for the total plant; an *indicated* treatment efficiency by the ponds, for removal of BOD_5 is 72%. The detention time afforded in the ponds (30 days or more) makes treatment efficiencies observed on a brief survey misleading. Some knowledge of the character of the pond influent a month or so earlier, when cannery loads were higher, would be necessary before a fair estimate of removal could be attempted. Long term oxidation of the pond effluent, however, more closely represents the extreme loading than BOD_5. In this instance BOD_5 for the pond effluent was only 21% of BOD_{30}. There is also evidence, in the studies of the rate of oxygen exertion, that the pond effluent, after acclimation of organisms, exerts a BOD at a rate substantially greater than the secondary effluent, a condition which could significantly intensify the oxygen sag near Stockton. This usually high demand rate could be due to acclimation of bacteria to the phytoplankton cell material of which more than 90% (numerically) are coccoid green algae, of which approximately 95% where of the genus *Scenedesmus*.

The Stockton Ship Channel was discolored by algae at various locations from the Turning Basin (see Figure 1) to the Terminus. Surveys of the study area with an instrument designed to measure *in situ* fluorescence showed consistently high chlorophyll fluorescence levels in the Turning Basin and the Terminus. Discolorations were also evident in the shallow river near the pond discharges; algal counts and recorded fluorescence confirmed visual observations of algae in this area.

Figure 8 illustrates variations in green algae (chlorophyta), number of cells and chlorophyll fluorescence noted in the study area. Upper river stations contain less than 20% coccoid green algae while downstream ship channel stations yield more nearly 50% green algae reflecting the discharge of these forms from the Stockton oxidation ponds. Total biomass, as indicated by *in situ* fluoresence at low slack tide, increased by slightly more than 50% downstream of the oxidation pond outfall as compared with upriver stations.

Oxygen demand measurements reflective of both algal respiration and bacterial oxygen utilization were made (under carefully controlled conditions of darkness to insure no photosynthetic oxygenation interferences). These tests included chemical oxygen demand (COD), biochemical oxygen demand (BOD), and organic carbon (C) and unoxidized nitrogen (N) forms.

Results of the above anlyses when compared with algal counts show the oxygen demand increases with increased algal crop. For example, the 50% increase in algal fluoresence measured near the discharge from the ponds was paralleled by comparable increases in BOD, COD and calculated theoretical ultimate oxygen demand (UOD) based on: $UOD = 2.67C + 4.57\ (Org\ N + NH_3\text{-}N)$ where C is the organic carbon and N is nitrogen (7). Table 6 contains data on oxidizable material loadings as calculated from flows of the river (1306 cfs) and oxidation ponds (32.9 mgd) during the survey. Detailed receiving water quality and waste water data are contained in a Federal Water Quality Administration Report. (2)

Chlorophyll *a* and COD measurements made in the study area in mid September yielded from 20 - 60 ug chloropyll *a* per liter and COD values from 15 to 30 mg/l; with peak values occurring in the pond discharge area. Ratio of COD to chlorophyll *a* were nearly constant through the area at ratios from 100 to 200 by weight; where bottom sediments were sampled this ratio rose to high values (ie 1700). Decomposition rates of pond algae appeared more rapid than natural river sample which

Table 5 - Summary of Flow of Waste Constituents Through the Stockton Wastewater Treatment Plant*

Constituent	Untreated Wastewater	Secondary Effluent	Pond Effluent
Total Nitrogen, mg/l (N)	27.2	18.1	25.9
Total Phosphorus, mg/l (P)	7.5	5.0	5.2
Total Carbon, mg/l (C)	328	232	209
BOD, mg/l (5 day)	578	340	96
BOD, mg/l (30 day)	822	460	458
COD, mg/l	870	660	575
Chlorinated Hydrocarbon Pesticides ug/l	2.70	1.15	0.34
Organophosphate Pesticides, ug/l	1.00	1.00	1.16
Fecal Coliform, MPN/100ml	60,000.	10	900
Phytoplankton, no./ml	--	--	211,000
Chlorophyll A, ug/l	--	--	270

*Average of four six-hour composite samples collected September 30-October 1, 1969.

Table 6 – Oxidizable Materials Summary*

Oxygen Demand Measurement	San Joaquin R. at Mossdale 1000 #/day	Water Quality Control Plant Discharge 1000 #/day
BOD_5	18	28
BOD_{29}	36	134
COD	84	166
UOD	143	164

*Based on observed waste and river flows during the October 1 1969 survey.

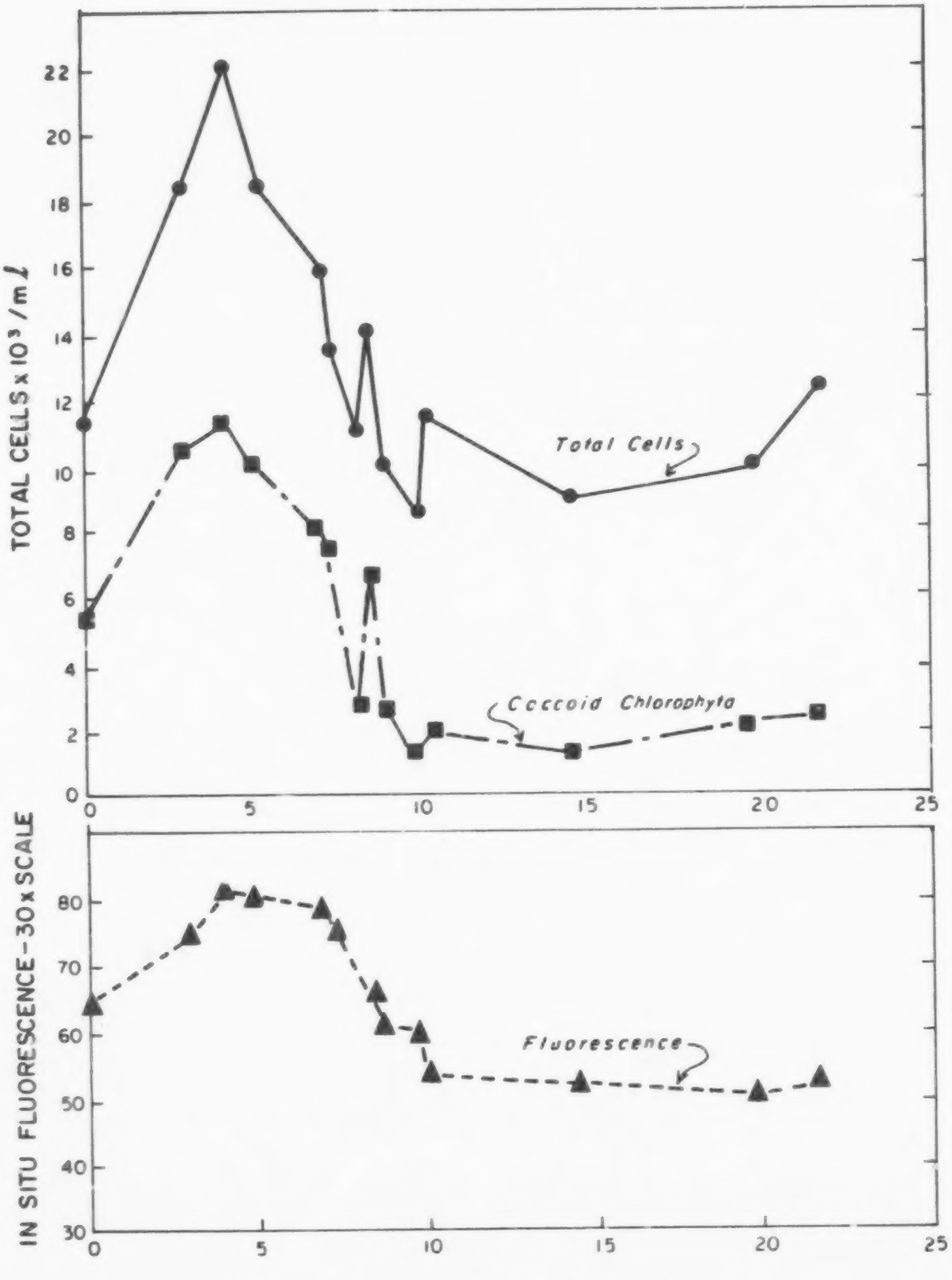

Figure 8. Comparisons of algal populations through study area using cell counts and chlorophyll fluorescence, September 30, 1969.

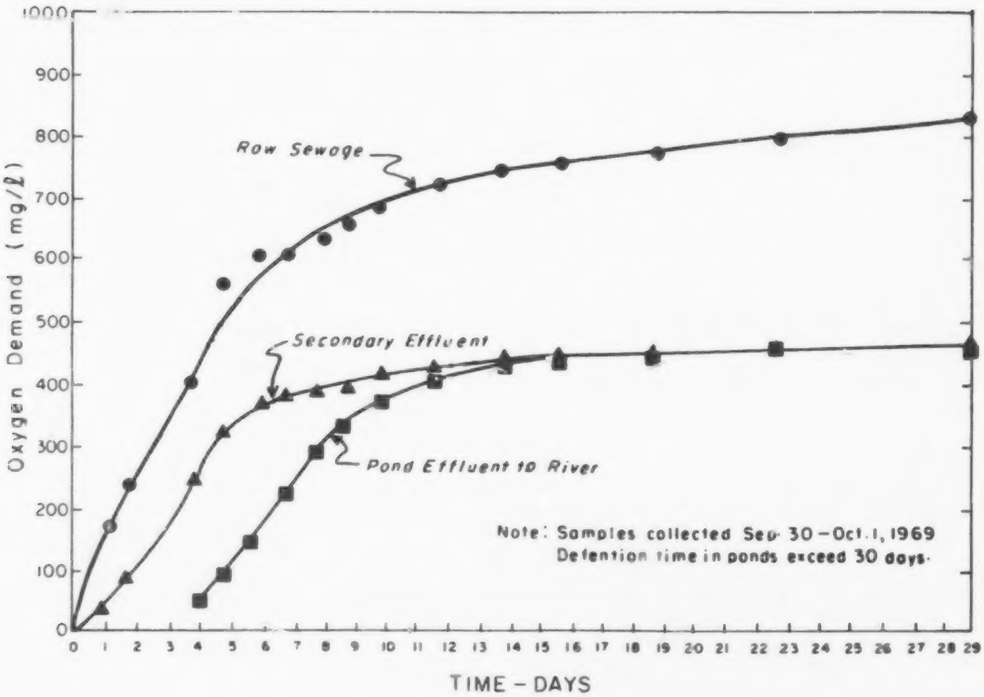

Figure 7. Oxygen demand from composite samples of raw sewage, secondary effluent (pond influent) and pond effluent.

appeared to contain refractory organic in soluble form; details on the relative decomposition of samples are contain in a report by Dr. P. L. McCarty. (8)

DISCUSSION

The Stockton ship channel has been progressively deepened at various times during this century, and the Corps of Engineers has recently been authorized to further deepen these waters to a depth of 35 feet. Deepening of the channel has markedly reduced the waste assimilative capacity of these waters. The reasons for this loss in assimilitive capacity relate to both physical and biological processes.

The physical factors directly contributory to the reduced assimilative capacity are the increased depth itself and the diminished tidal velocity caused by the larger channel cross section. Both factors tend to depress atomspheric reoxygenation.

Reductions in velocity or increases in depth lead to lower oxygen transfer rates in stream values, implying a lower transfer of oxygen from the air to waters of depressed oxygen content.

The most important factor is biological, since the deepended channel serves as a stilling basin to collect and retain organic material entering from the river. Much of this organic material is living algae grown in the shallow, war, well lighted, nutrient enriched river and in the large sewage oxidation ponds operated by the City of Stockton. Algal growth and photosynthetic oxygenation are efficiently carried out in the shallow river and in the ponds, but the deep turbid ship channel does not permit light energy to penetrate through enough of the water column to maintain the algal populations which are eventually trapped in the channel. As a consequence, the populations decline and respiratory demands of algae, zooplankton and bacteria prevail leading to frequent and sustained oxygen depressions in the lower depths during the summer and fall.

The natural flora of the San Joaquin River are primarily diatoms. The "Natural" river flora is altered in the vicinity of the Stockton sewage treatment plant by high concentrations of other algal groups principally the coccoid green algae. After the river flow was increased in 1964 the green algal buildup near the pond discharge shifted downstream and was diluted.

What is the fate of these algal organisms? A balance of the number of coccoid green algae discharged from the pond and populations in the river above a base level set in 1964 by cruise 1 (See Figure 5) indicates that, at best, this type of organism survives in a recognizable form in the river for only one or two days. The green algae may be consumed by grazing zooplankton, settle and decompose on the river bottom or respire and decompose in the turbid ship channel waters. An analysis of inert diatom shells in ship channel stations indicated that dead cells make up a higher percentage of the total population near the bottom as compared with the surface samples. (1)

Zooplankton populations were measured by the California Department of Fish and Game during the 1964 study period. Populations in the ship channel were at least 100 times as great as at Mossdale Bridge (Station 7) farther up river. The zooplankton netting methods employed did not pick up all grazing organisms, but yielded principally the larger adult animals (including larger rotifers).

The impact of flow depletions and resulting flow reversals and lengthened residence times in the Stockton channel are apparent considering the results of the flow releases and the impact of deoxygenation processes in the deep water channel. These processes are further aggravated by water temperature increases which increase respiratory demands while dissolved oxygen saturation values fall. The combined effect can be expected to produce lower *in situ* DO values in the channel although organic loadings (as measured by BOD or COD) may remain the same. To further aggravate the picture, photosynthetic contributions can be expected to wane as summer wears on since the hours of daylight diminish from the vernal maximum.

Thus, two major factors affecting oxygen demand exerted within the study area are loading of oxygen-demanding materials and time of travel. Loading of oxygen-demanding materials is mainly from two sources: the City of Stockton's Main Water Quality Control Plant and the upstream reaches of the San Joaquin River. Time of travel depends on the volume of inflow, as shown in Figure 3. Autumnal flows in the reach range from less than 100 cfs over 1000 cfs with the October average over the past five years near 550 cfs. At calculated retention times for this flow the organic loadings from the oxidation pond during the canning season exceed that from the river upstream.

SUMMARY

The information gathered during the course of these investigations serve to put the several factors affecting dissolved oxygen in the Stockton Ship Channel in perspective. Flow augmentation as attempted in 1964 tended to force low quality water downstream rather than upgrade the channel oxygen supply. The deep ship channel itself retains pollutants for long periods; further deepening will reduce waste assimilative capacity of these waters. Algae and other organic matter entering the channel area from the river upstream and from oxidation pond effluents do not thrive in the deep, turbid channel where respiratory oxygen demands exceed photosynthetic oxygenation. The oxygen demand loading from the oxidation ponds operated by the City of Stockton generally exceeds that of the San Joaquin River upstream during the late summer-early autumn period.

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MATHEMATICAL SIMULATION OF WASTE STABILIZATION PONDS

*Joseph F. Roesler and Herbert C. Preul**

INTRODUCTION

Many excellent mathematical relations have been developed for use in simulation or in design of waste stabilization ponds. Unfortunately, the existing relations lack sufficient detail to totally describe the activities occurring in the pond or else their application is limited to a few specific types of ponds.

The model for the waste stabilization pond of this paper is based on an influent of domestic sewage. It is assumed that some of the sewage that enters the pond water settles and forms a shallow sludge layer. BOD within the sludge layer is degraded by anaerobic processes yielding 1) gaseous products and 2) a soluble BOD product which enters the pond waters. The mechanism for BOD degradation in the pond water is assumed to be aerobic. In

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actual practive it would be difficult to obtain such distinct classification of mechanisms. The pond would be facultative with predominately anaerobic processes occurring in the bottom layers and aerobic processes at the top. Consequently, the mathematical model described herein is assumed to apply to facultative poinds, also.

In actual practice, some ponds may employ conventional primary treatment, or the raw sewage may flow into anaerobic or facultative ponds without primary settling. Aerobic ponds are generally shallow in depth and are used for final or finishing treatment. Breaking ponds up into smaller cells appears to improve the efficiency of BOD removal. Some ponds depend heavily on algae growth and may employ recirculation as a mixing device. Finally, some ponds are designed to discharge effluents while others may depend heavily on evaporation and seepage for water removal.

THEORY

Many complex biological processes occur simultaneously in a waste stabilization pond. To characterize mathematically all processes would be difficult, if not impossible. However, if one considers the pond in terms of gross parameters, a mathematical solution is possible. Reaeration is an example. Bacteria utilize the BOD within the pond, grow in number and, after death and lysis, settle out in the sludge layer. The limiting factor in this chain of events is the oxygen content, and a gross parameter that may be used for mathematical simulation is the reaeration rate.

Algae act similarly; however, they can produce oxygen which would influence the reaeration rate. The algae effect is also a gross parameter that can be described mathematically. Finally, the anaerobic process occurring in the sludge layer is a gross parameter for which a mathematical description is proposed herein.

BOD values were used to measure the concentration of substrate largely because the literature contained detailed data related to BOD measurements. While it can be argued that BOD is not the best measure of the effluent quality because of algae interference,[1] the BOD test is still a reasonably good estimate of the influent water quality. And a 50% measurement error due to algae in the effluent BOD for a pond removing an actual 80% of the BOD would result in an increase of apparent removal to 90%.

Because of the long detention time of ponds the ultimate BOD, S_u, (mg/1) appears to be a better description than the 5-day BOD, S, (mg/1). These two BOD's are related by

$$S = S_u\ (1\text{-}e^{-kt})$$

where k is a constant and t is the time which in the case of 5 days, t = 5. Assume that the influent BOD, flow and climate are constant over a period as long as the detention time, then, $S = K\ S_u$ where K is a constant. Due to this simple relationship, 5-day BOD's are used in the following discussions.

Aerobic Section of the Pond

The first mathematical expression derived in this section simulates the gross overall activity of BOD as it enters the pond.

Consider the pond with a volume, V, and influent flow, Q, and an effluent flow, Q_2. The difference, $Q - Q_2$, is Q_3 and is defined as the volume of water lost per unit of time due to evaporation and seepage. Assume that every pond has a sludge blanket on the bottom. This blanket acts as a slow sand filter resulting in negligible BOD loss due to seepage. Assume also that the BOD remaining in the pond is removed by first order kinetics.

$$\frac{dS}{dt} = -KS$$

where K is the first order rate constant.

To determine what is happening in the whole pond a mass balance is necessary.

$$dS = (\frac{Q}{V})\ S_i\ dt - (\frac{Q_2}{V})\ S\ dt - K\ Sdt$$

where S_i is the 5-day BOD entering and S is the 5-day BOD in the pond volume, mg/1.

Since $Q_2 = Q - Q_3$, the equation above may be rearranged to

$$\frac{dS}{dt} = \frac{(Q)}{V}\ S_i - \frac{(Q - Q_3)}{V}\ S - KS \qquad 1)$$

The retention time, R shall be defined as $\frac{V}{Q}$. It is reasonable to assume that in a pond, steady state conditions exist, thus $dS/dt = 0$ and (1) becomes

$$\frac{S_i}{R} = \left(\frac{1}{R} - \frac{Q_3}{V}\right)\ S + KS$$

simplifying

$$S_i = (1 - \frac{Q_3}{Q} + RK)\ S \qquad 2)$$

In an aerobic pond system there will be some sludge that settles and compacts. There will be a deficiency of oxygen in the compacted sludge. It is assumed that BOD degradation will take place in the sludge by anaerobic processes. Marais[2,3,4] described the degradation of the BOD in the sludge layer by the series of equations below. The total BOD, S_{ss}, (mg/1), in the sludge varies according to the differential equation:

$$\frac{dS_{ss}}{dt} = K_s P_{ss} + f_s\ S_i Q$$

where K_s is the temperature-dependent constant relating to the sludge, S_i is the BOD entering the pond at a flow of Q, and f_s is the fraction of the influent BOD settling out as sludge.

Marais stated that:

the change of BOD in the pond volume = the fraction of influent BOD that remains suspended +

amount of BOD that regurgitates from the pond bottom + reduction of BOD in the pond volume

or mathematically stated:

$$\frac{dS}{dt} = \frac{f_p S_i}{R} + \frac{K_s\ C_p\ S_{ss}}{V} + (1 + RK - \frac{Q_3}{Q})\ S$$

where f_p is the fraction of influent BOD that is dispersed within the body of the pond.

At equilibrium $dS/dt = dS_{ss}/dt = 0.0$

Combining all the above relations the concentration of BOD in the sludge is

$$S_{ss} = \frac{f_s\ S_i\ Q}{K_s}$$

and in the pond

$$S = \frac{S_i}{1 - \frac{Q_3}{Q} + RK}\ (f_p + C_p\ f_s) \qquad 3)$$

The ratios f_p, f_s, and C_p are defined as:

$$f_p = \frac{S_d + S_{so}}{S_i}$$

$$f_s = \frac{P_{ss}}{8.34}\, S_i\, Q$$

$$C_p = \frac{P_{sr}}{P_{ss}}$$

when S_d is dissolved 5-day BOD entering pond, mg/1; S_{so} is suspended solid 5-day BOD in pond that refuses to settle, mg/1; S_i is total 5-day BOD entering pond, mg/1; P_{ss} is solid BOD in the sludge layer, lbs/day; P_{sr} is BOD returning to the pond volume due to fermentation, lbs/day. (Note that all BOD symbols of the letter S, have units of mg/1, while all letters P have units of lbs/day.)

Substituting the above relations into equation (3) yields

$$S = \frac{S_i}{\left(KR - \frac{Q_3}{Q} + 1\right)} \left[\frac{S_d + S_{so}}{S_i} + \frac{P_{sr}}{P_{ss}} \frac{P_{ss}}{S_i\, 8.34Q}\right]$$

Simplifying

$$S = \frac{(S_d + S_{so})\ 8.34Q + P_{sr}}{8.34\ \left(KR - \frac{Q_3}{Q} + 1\right)} \qquad 4)$$

If S_{sr} is defined as the BOD returning to the pond volume due to fermentation in units of mg/1 and, S_x is the sum of S_{sr} and S_{so}, then,

$$P_{sr} = S_{sr}\ 8.34Q$$

$$S_x = S_o + S_{sr}$$

Substituting into equation (4) yields

$$S = S_o = \frac{S_d + S_x}{\left(KR - \frac{Q_3}{Q} + 1\right)} \qquad 5)$$

where S_o is the pond effluent.

In the Data Analysis section, K was found to be .0423 ± .0476 at 95% confidence limits based on the analysis of 98 sets of data.

Mass Balances About a Pond

Another method of viewing the gross activities occurring in a pond is to make a mass balance of materials entering and leaving. All the mass balances for BOD, P, are in units of lbs/day. Figure 1 is a schematic representation of the BOD distribution in the pond volume. Starting at the left of the Figure, P_i is the influent BOD entering the pond which is composed of the dissolved BOD entering, P_d, and the solid BOD entering P_s. The amount of solid BOD that settles is P_{ss} while the amount that remains in suspension is P_{so}. Of the BOD that settles P_{ss}, a certain amount will decompose anaerobically, P_g, liberating methane gas. This breakdown of solids will probably result in generating some soluble BOD, P_{sr}, which will escape the sludge layer and rise into the pond volume. Some BOD will settle and not degrade at all, P_n. The effluent BOD is P_o.

Note that in Figure 1 all the material in the pond volume $P_d + P_{so} + P_{sr}$ is assumed to be degraded by aerobic process and therefore equation (5) applies

$$P_d + P_{so} + P_{sr} = P_d + P_x = P_o \left(KR - \frac{Q_3}{Q} + 1\right)$$

Recalling that $P_x = P_{so} + P_{sr}$

There are two mass balances that can be made. The first is a balance on the BOD entering the pond and the second on the BOD consumed in the pond or leaving the pond.

These mass balances are represented below.

1) BOD entering the pond

$$P_I = P_d + P_{so} + P_{sr} + P_g + P_n \quad \text{or}$$

$$P_I = P_d + P_x + P_g + P_n \qquad 6)$$

2) BOD removed

$$\begin{array}{c}\text{BOD}\\\text{entering}\end{array} = \begin{array}{c}\text{BOD in}\\\text{effluent}\end{array} + \begin{array}{c}\text{BOD removed}\\\text{by reaeration}\end{array} + \begin{array}{l}\text{BOD removed}\\\text{by anaerobic}\\\text{processes}\end{array} + \begin{array}{l}\text{BOD that settles}\\\text{to pond bottom}\\\text{undecomposed}\end{array}$$

$$P_i = P_o + P_a + P_r + P_g + P_n \qquad 7)$$

where P_a is the BOD removed by algae, lbs/day, P_r is the

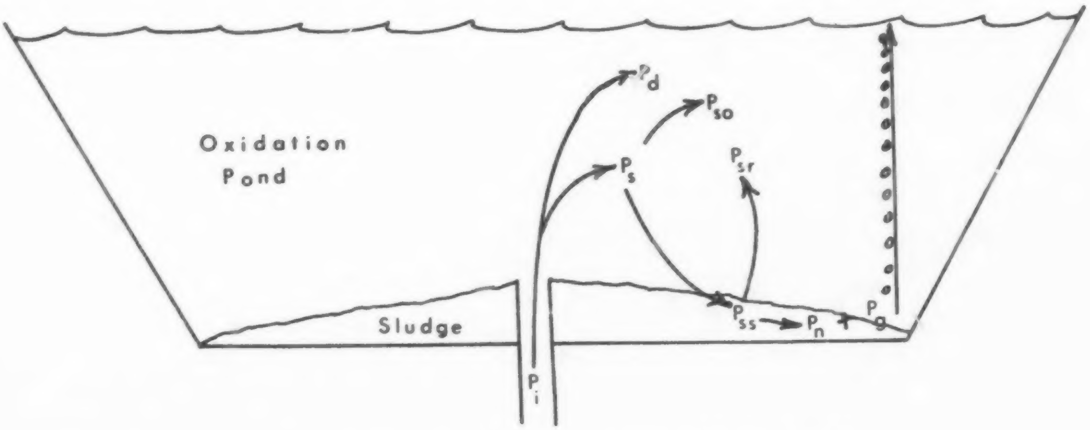

Figure 1. Schematic representation of BOD distribution in pond volume

BOD removed by reaeration, lbs/day. One can easily see that if all computations above are based on the flow through the pond the units of lbs/day would be readily converted to mg/l. Thus, equations (6) and (7) become

$$S_i = S_d + S_x + S_g + S_n \qquad 8)$$

$$S_i = S_o + S_a + S_r + S_g + S_n \qquad 9)$$

BOD Removed by Reaeration

Imhoff and Fair[5] found that for the pond the BOD removed by reaeration is

$$R_a = .0271\ (20)\ (D_f)\ (D) \qquad 10)$$

where R_a is the BOD removed due to reaeration in lbs/acre/day, D_f is the DO deficit, mg/l, D is the aerobic pond depth, ft.

The saturation value for the dissolved oxygen, D_{os}, in fresh water was taken from the data of Whipple and Whipple[6]. A regression fit is

$$D_{os}\ (\text{mg/l}) = 14.652 - .41022\ (T) - .007991\ (T)^2 - .00007774\ (T)^3$$

where T = temperature in °C. If the dissolved oxygen, D_o, is known, the DO deficit can be calculated:

$$D_f = D_{os} - D_o$$

If the area of the pond is A (acres) and BOD removed based on total flow through the pond, then:

$$S_r = \frac{R_a\ A}{8.34Q} \qquad 11)$$

By definition the retention time, R, (days) is equal to

$$R = \frac{V}{Q}$$

where V = the volume, millions of gallons

Since volume is a function of area and depth, then

$$R = \frac{AD}{3.068Q} \qquad 12)$$

where A is area in acres, and D, depth in feet

Substituting equations (10) and (11) into (12) yields

$$S_r = .1994\ (D_f)\ (R) \qquad 13)$$

BOD Removed by Algae

Oswald[7] has proposed a model for high-rate aerobic ponds that is linked to algae production. High-rate aerobic ponds are shallow ponds, with recirculation and designed to have a vigorous algae growth. The algae produce oxygen to oxidize sewage which in turn acts as an organic substrate for further algae growth. While his equations and data relate to high-rate ponds, there are several items that can be applied to algae growth in general.

Empirically, Oswald has related the concentration of algae, C_c, (mg/l) to the BOD removal, S_a, by a factor of 1.5.

$$C_c = S_a/1.5 \qquad 14)$$

The depth of light penetration, D', is defined by Beer-Lamberts law as

$$D' = \frac{\ln\ (I_o/24)}{.0015\ C_c} \qquad 15)$$

The extinction coefficient is given by Oswald as .0015. I_o is the radiant intensity in foot candles. A detailed description on how to estimate I_o may be found in the work of Oswald[8]. If the algae species were *Chlorella pyrenoidosa*, then the minimum intensity necessary to sustain life would be 24 foot candles according to the work of Phillips and Myers[9]. The depth, D', calculated above relates only to the algae depth. Since S_s is defined as based on the entire pond volume, S_a in equation (14) must be corrected to reflect the dilution. Thus

$$S_a = \frac{1.5\ C_c D'}{D} \qquad 16)$$

substituting D' of equation (15) in (16) yields

$$S_a = \frac{32.808\ \ln\ (I_o/24)}{D} \qquad 17)$$

Oswald[7] supplied numerous data on the temperature effect on the efficiency factors for BOD removal. A polynominal curve fit was applied to his data resulting in

$$T_c = -.00328\ T^2 + .1467\ T - .6363 \qquad 18)$$

where T_c is the temperature correction. This equation is valid for temperatures for 4° C to 32° C. Considering temperature, equation (17) becomes

$$S_a = \frac{32.808\ T_c\ \ln\ (I_o/24)}{D} \qquad 19)$$

Anaerobic Decomposition and Equations for Depth

In any aerobic pond, there will be settling of solids which will eventually increase in depth on the bottom of the pond, become more compact and deplete the supply of dissolved oxygen. In this sludge layer, then, anaerobic degradation will begin. Some of products are methane and lower molecular weight and more soluble organics. The methane will escape as P_g and the soluble organics will return to the body of the pond as P_{sr}.

There will be sludge accumulation, but the amount of sludge accumulation appears to be small. Middlebrooks, Panagiotou, and Williford[10] measured the sludge accumulation in several ponds and found from 4 to 7 inches of accumulation after 82 months of operation.

Oswald[11] measured the gas evolved from ponds using a Bronson gas collector with a bottom area of 4 ft.^{2}. Using data from his report a multiple regression analysis yielded the resulting equation given below, with a multiple correlation of .87 for 31 sets of data.

$$G = .17023 \times L^{1.098} \times 1.163^T$$

where G is the total gas evolved in ft^3/acre day, and L is BOD loading in lbs/acre day. However, only about 60% of G is methane gas[11]. McCarty[12] stated that one pound BOD is equivalent to 5.62 ft^3 methane at standard temperature and pressure. Thus, making all the corrections to standard conditions and only 60% methane yields

$$\frac{\text{lbs BOD degraded}}{\text{acre day}} = \frac{4.961\ (L)^{1.098}\ (1.163)^T}{(273 + T)} \qquad 20)$$

Recalling equation (7) the amount which accumulates in the sludge is

$$P_g + P_n = P_i - P_o - P_a - P_r$$

Multiplying both sides of the equation (20) by the area, A, (acres) and using equation (7) yields,

$$P_g = \frac{4.961\ (P_g + P_n)^{1.098} (1.163)^T}{(273 + T)\ A^{.098}}$$

Converting P_g to units of S_g gives

$$S_g = \frac{6.1078\ (S_g + S_n)^{1.098}\ 1.163^T}{(273 + T)} \left(\frac{Q}{A}\right)^{.098} \qquad 21)$$

The depth of an aerobic pond was taken from data supplied by Oswald[13]. These data were observed to fit a straight line or a semi-log plot. The resulting equation is

$$\ln D = 2.299 - .02032\ L$$

where L is the BOD loading in lbs/acre day

$$L = \frac{S_i\ Q\ 8.34}{A}$$

where the ratio $\frac{Q}{A}$ is $\frac{D}{R}$

substituting the above

$$\ln D = 2.299 - .05546\ S_i\ D/R \qquad 22)$$

Equation (22) is unsolvable by conventional techniques. The Newton-Raphson numerical technique was used to obtain a solution.

Interrelationships Between Equations

If equations (7) and (8) are equated and $S_d + S_x$ determined from equation (5) the result is

$$S_o \left(KR - \frac{Q^3}{Q}\right) = S_a + S_r \qquad 23)$$

but

$$Q_3 = .027\ E\ A = .0833\ \frac{E\ Q\ R}{D}$$

where E is the evaporation and seepage loss in inches/day, and A is the area in acres. Substituting into (23) yields

$$D = \frac{32.81\ T_c\ \ln (I_o/24) + .0833E\ S_o\ R}{(S_o\ K - .1994\ D_f)\ R} \qquad 24)$$

The unknowns in equation (22) and (24) are now R and D, hence the two equations can be solved. If the conditions are such that no algae will grow, equation (24) simplifies to

$$D = \frac{.0833E\ S_o}{(S_o\ K - .1993\ D_f)} \qquad 25)$$

and the retention time R can be calculated from equation (22).

DATA ANALYSIS

Data Analysis

The theory of the previous section is to be evaluated with experimental data. For such an evaluation, it would be necessary to know the effluent BOD, detention time, depth, sunlight intensity, temperature, evaporation rate, and dissolved oxygen deficiency. The constant K of equation (5) could then be evaluated and an estimate of the applicability of the method could be obtained from the standard deviation of the values for K. A computer program was designed to calculate the pond constant, K, and the disposition of all the BOD's. Experimental data were obtained from the literature and included ponds located in the Dakotas[14], Mojave, California[15], Fayette, Missouri[16], and Canada[17].

Ninety-eight items of data were analyzed. The average value of the pond constant was .0423 with a standard deviation of .0238. The values ranged from .0008 to .1338. The standard deviation may be improved if a better judgment could be made as to which BOD effluent corresponds to a given influent BOD when one considers the long detention time of ponds. Nevertheless, the authors felt that the standard deviation was sufficiently low to justify the constant K and, of course, the theory of the previous section. A variation of the above computer program related the constant K to the temperature and revealed that no further temperature correction was necessary.

Costs

There are several difficulties in obtaining cost information for oxidation ponds. First, land costs vary as to location, land uses and human factors. Secondly, the cost of construction varies as to the degree of mechanization desired, type of soil, and the local labor market. Some communities with non-porous type soil require a polyethylene liner. Finally, lagoons are generally used when simplicity of treatment is desired; as a result, adequate records of operation and maintenance costs are lacking.

The capital cost used in the program was the data of Rowan, Jenkins, and Butler[18] updated to February 1970 by the FWQA construction cost index.

$$\text{Cost (dollars)} = 109400\ (Q)^{.744}$$

Recently, the State of California Water Resources Control Board[19] published itemized costs for lagoon construction. The report is detailed but the costs are presented only for a typical case of an 8 mgd flow. Regression equations for other flows were also developed but, in general, these flows were more than those of Rowan, et al. costs. The operating and maintenance costs were also fitted by regression equations. Using these regression equations, the data was recalculated for 85% BOD removal at various flows. A new regression yielded

$$\text{Costs (dollars)} = 9659\ (Q)^{.633}$$

in terms of February 1970 dollars.

DISCUSSION AND CONCLUSIONS

The mechanism of BOD degradation in a pond is complex, involving both aerobic and anaerobic processes. In aerobic processes a host of organisms will utilize BOD as reflected by an increased reaeration rate, but very little is known about the anaerobic processes. Algae grow utilizing BOD and generate oxygen for the other organisms to utilize BOD.

The mathematical relations described for both the algae contribution and the reaeration rate are approximations, and were used in lieu of more accurate relations to describe the activities within aerobic-anaerobic pond systems. Oswald's algae equations relate largely to a special type of pond. Their utility of application has been extended to give better insight into the activities going on within a pond. The algae contribution as used in this work is dependent on the temperature, light intensity and depth. The intensity of light is dependent on the solar insolation, the latitude, time of year and percent cloudiness. Consequently, the algal relationships used would be applicable in amy climate and latitude.

While it is true that there are numerous existing formulations for atmospheric reaeration, such formulations do not apply to waste stabilization ponds. The approximate empirical equation of Imhoff and Fair[5] was assumed to fit. As the temperature decreases the "driving force" (oxygen deficit) for reaeration and the reaeration rate are increased according to Imhoff and Fair. The ability of the microorganism to utilize the oxygen at the colder temperature is not considered. The suggestion has been made that reaeration should take into account the surface agitation, exposure of fresh water surface, and/or mixing. Phelps[20] noted that in New York Harbor, a wind velocity of as little as 5 miles per hour would generate a depth of uniform mixing up to 10 feet. In open areas such as areas where ponds are usually found, an average wind of about 5 miles per hour would be expected over a long period of time. Consequently, it was assumed that ponds described here were continually in a state of surface agitation and, as a result, had excellent reaeration rates.

One of the conclusions of the present work is that the importance of algae contribution has been overemphasized. Other processes, reareation and anaerobic, are equally, if not more important. The fact that little sludge accumulates in ponds should demonstrate the importance of the anaerobic activity. Furthermore, in pond design, the area should be made sufficiently large to guarantee a year-round aerobic depth. The actual depth of the pond may be made much deeper than the aerobic depth to accomodate anaerobic processes.

The chief advantage of ponding is that the costs are less than other processes accomplishing the same treatment. Ponds are ideally suited to small communities. Consider a typical example of a small pond (.01 mgd) designed for 85% BOD removal. The equations in this paper indicate that the pond requires .64 acres of area and a depth of 4.3 ft. The construction cost is $3560 and the maintenance costs $523 per year. The total cost of treatment (22 cents per 1000 gallons) is high, but when compared to most other forms of treatment, the cost is still small.

BOD removal efficiencies for ponds are usually on the order of 60 to 96%. The California Report[19] limits the BOD removal efficiency for aerobic ponds between 68 and 96%, for facultative ponds between 60 and 90%, and for anaerobic ponds between 50 and 80%. There are times when the algae do interfere and appear in the effluent to reduce the overall removals. Perhaps this problem could be somewhat averted by designing deeper ponds and/or controlling the level at which the effluent is taken so that the algae layer is left intact. Deeper ponds would probably increase the anaerobic activity, thereby decreasing the amount of algae. Additional knowledge of the anaerobic processes occurring would be helpful.

DEFINITIONS OF ALGEBRAIC SYMBOLS

Symbols	Definitions and Comments
A.	Area of pond, acres
C_c	Concentration of algae, mg/1
C_p	Fraction of BOD in sludge that enters pond volume
D	Aerobic depth, ft
D'	Depth of light penetration, cm
D_a	Anaerobic depth, ft
D_f	Dissolved oxygen deficit, mg/1
D_I	Diurnal illumination factor
D_o	Dissolved oxygen concentration, mg/1
D_{os}	Saturation value for dissolved oxygen, mg/1
E	Evaporation and seepage loss, in /day
f_s	Fraction of the influent BOD settling out as sludge
f_p	Fraction of the influent BOD dispersed within the body of the pond
G	Total gas evolved, ft^3/acre day
I_o	Radiant intensity, ft candles
K	Constant, day^{-1}
K_s	Constant
L	BOD loading, lbs/acre day
P_g	BOD removed by anaerobic processes, lbs/day
P_o	Effluent BOD, lbs/day
P_n	BOD that settles to pond bottom, lb/day
P_r	BOD removed by reaeration, mg/1
P_{sr}	BOD returning to the pond volume due to fermentation, lbs/day
P_{ss}	Total BOD in the sludge layer, lbs/day
Q	Influent flow, mgd
Q_2	Effluent flow, mgd
Q_3	Volume of water lost due to evaporation and seepage, mgd
R	Retention time, days
R_a	BOD removed by reaeration process, lbs/acre/day
R_n, R_T	Values for retention time used in iteration
S	5-day BOD in body of pond, mg/1
S_a	BOD removed by algae processes, mg/1
S_d	Dissolved 5-day BOD entering pond, mg/1
S_g	BOD removed by anaerobic processes, mg/1
S_i	Influent 5-day BOD, mg/1
S_n	BOD that settles to pond bottom, mg/1
S_o	Effluent BOD, mg/1
S_r	BOD removed by reaeration processes, mg/1
S_{so}	Suspended solid 5-day BOD in pond water that refuses to settle, mg/1
S_{sr}	BOD returning to the pond volume due to fermentation, mg/1
S_{ss}	Total BOD in sludge, mg/1
S_T	Total ultimate BOD in sludge, mg/1
S_u	Ultimate BOD in pond, mg/1
S_{ui}	Ultimate influent BOD, mg/1
S_x	$S_{so} + S_{sr}$, mg/1
t	Time, days
T	Temperature, °C
T_c	Temperature factor for algae growth
V	Volume of pond, acre-ft

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DESIGNING WASTE PONDS TO MEET WATER QUALITY CRITERIA

William J. Oswald, Aaron Meron, and Mario D. Zabat[**]

INTRODUCTION

The list of criteria used by research workers, treatment plant designers and operators, and regulating agencies to evaluate water quality is increasing in length and complexity. Criteria such as plaque-forming units for viruses, median tolerance limits for toxicity, and algal growth potential for nutrients have emerged while venerable parameters such as relative stability have fallen into disuse. Although the objective in pond design is to attain acceptable levels for water quality criteria, current designs more often than not are made on the basis of rules of thumb developed over a number of years in a given geographical area. This sometimes leads to overdesigned systems, but more often to underdesigned systems. The question then arises: Can ponds be rationally designed to meet rigorously specified discharge requirements or must they be designed by rule of thumb and relegated to the category of essentially uncontrolled holding or dilution bodies? The answer, as we have emphasized in several papers (1, 2, 3), is that ponds can indeed be designed to meet specific water quality criteria, provided environmental and ecological factors are taken into consideration. In this paper we will present further evidence of the rationality of pond disign and its successful application in meeting water quality standards.

In a recent NAS/NRC report on waste management and control (4) pollutants entering water courses were broadly classified in eight categories: (sic) 1) Domestic sewage and other oxygen-demanding wastes, 2) infectious agents, 3) plant nutrients, 4) organic chemicals such as insecticides, pesticides, and detergents, 5) other minerals and chemicals such as chemical residues, petrochemicals, salts, acids, silts and sludges, 6) sediments from land erosion, 7) radioactive substances, and, 8) heat from power and industrial sources.

The above categorical list will be utilized as a basis for our consideration of design factors for ponds to meet water quality criteria.

OXYGEN-DEMANDING WASTES

The possibility that ponds could be utilized as a mechanism for removal of biochemical oxygen demand has been recognized for nearly 50 years (2). Algal growth with concomitant oxygen production has been credited with satisfaction of BOD, but in reality there is a transformation of a variable fraction of the original sewage organic matter to algal organic matter so that, as pointed out by Barsom (5), occasionally in raw sewage ponds, the effluent COD may exceed that of the influent. Parker (6) was the first to document the importance of anaerobic ponds in sewage stabilization, and Bronson (7) through in situ measurement of methane emission, clearly established methane fermentation as the major mechanism of BOD removal in ponds. Methane fermentation is, of course, highly sensitive to temperature and, as documented by Hermann (8), temperature is therefore an important factor in any pond design. As pointed out in the first Kansas City Conference on ponds (9), BOD may be stored, it may be converted to CO_2, organic acids, methane, or it may be converted to algae which, in turn, may settle and accumulate, undergo methane fermentation or be discharged.

Table 1 shows the reduction in supernatant BOD in a single undisturbed pond as a function of temperature. This reduction results from a complex of factors including sedimentation, oxidation, fermentation, and conversion to algae and other organisms. The reduction in supernatant suspended solids in ponds is initially a physical phenomenom in which more than 50 percent of the suspended solids settle in one-half hour or less (see Table 2). If the environment is satisfactory at the bottom of the pond, methane fermentation of the settled solids occurs. The requirement for adequate methane fermentation is protection of the fermentation zone from the intrusion of oxygen and assurance that temperature above 15°C is maintained in the fermentation zone. The relationship between methane fermentation and temperature is that:

$$G = 450\,(T - 15) \qquad (1)$$

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Table 1 – Natural Reduction in Supernatant BOD in an Undisturbed Pond*

Time in Days	Percent BOD Removal at Indicated Average Temperature			
	10°C	14°C	15°C	18°C
0	0	0	0	0
1	23	35	47	33
2	40	61	56	53
4	46	70	67	56
9	57	75	75	81
16	72	89	90	87
32	90	90	95	96
64	94	--	94	--

*From Ref. 3

Table 2 – Reduction in Supernatant Suspended Solids at a Depth of 12 inches Due to Sedimentation in a Quiescent Facultative Stabilization Pond.

Time in hrs.	Suspended Solids mg/liter	% Reduction
0	225	0
0.25	120	47
0.50	96	57
1.00	90	60
2.00	84	63
4.00	60	73
8.00	44	81
24.00	33	85
48.00	29	87

in which G is the gas production in ft^3/acre per day and T is the temperature in °C. (This differs from that in our earlier publication (7) by a factor of about one-half because of a numerical error in calculations of the area of Bronson gas collectors). If a conversion of 8 ft^3 of gas per lb. of BOD is assumed, methane fermentation may lead to elimination of from 56 lbs. of BOD per acre per day at a temperature of 16°C to 1120 lbs. per acre per day at a temperature of 35°C. As indicated in a previous paper (3), the temperature of a pond if taken in summer often declines with depth. In several lightly loaded facultative ponds, the mean lapse rate for change in temperature with respect to depth above the thermocline is about one-half degree centigrade per foot of depth, thus for quiescent ponds the deeper a pond the lower (although more stable) will be its bottom temperature. On the other hand, ponds which are undergoing a vigorous methane fermentation become mixed as a result of rising gas bubbles and tend to have a uniform vertical temperature profile. Heat absorbed at the surface becomes commingled within the entire hydromass and, hence, in ponds loaded sufficiently that oxygen does not interfere, thermal energy from the sun benefits fermentation. Few odors are evolved when methane fermentation is fully active, although the fear of odors is a major deterrent to use of anaerobic ponds.

In the case of facultative ponds under quiescent conditions, a point will be found below which there is no free dissolved oxygen. This point which varies in depth is termed the oxypause (3). The change in oxygen with respect to depth and hence the position of the oxypause is a function of relative rates of photosynthesis and respiration in the prism of water under consideration. Under average climatic conditions, a pond loaded at the rate of 200 lbs. of BOD per acre per day will have a surface layer containing dissolved oxygen and anaerobic conditions below. During mid-summer, a loading of 400 lbs. per acre per day may be accompanied by superficial layers containing dissolved oxygen, but during the winter, loadings of 100 lbs. per acre per day may deplete the surface of free dissolved oxygen. Above the oxypause, photosynthesis and aerobic oxidation occur and photosynthesis exceeds repiration. Below the oxypause, aerobic processes cease, and anaerobic processes such as organic acid fermentation and methane fermentation prevail. It is known that pond size greatly influences the depth of oxypause, the pause occurring at greater depths in large ponds. However, the precise relationship between pond size and performance remains to be determined. The intensity of photosynthesis also influences the oxypause, a shallow oxypause accompanying the highest rates of photosynthesis.

If BOD removal is a primary objective of pond design, it is essential to provide for carbon removal through methane fermentation or to provide for conversion of carbonaceous material to algae with subsequent provision for removal of the algal cells. Methane fermentation is by far the least expensive of the two processes and often leads to an effluent suitable for discharge without algae removal. This is because algae themselves may in part be induced to settle and undergo methane fermentation and denitrification.

To take these many factors into consideration for pond design to attain specific BOD removals, one should enter Table 1 with the required effluent BOD and the low mean temperature and determine the time required to attain the required effluent BOD at the temperature in question. The pond should then be designed with sufficient depth to protect the benthal layers from the intrusion of dissolved oxygen so that methane fermentation can proceed. This is easily done by providing a depth of at least 5 feet in small ponds, but in large ponds subject to wind mixing, it appears that special pockets should be provided to permit accumulation of sludge and methane fermentation to occur. Seeding with actively fermenting methane sludge will accelerate onset of this process. If algae are unacceptable in the effluent, a second cell or settling pond may be provided which is also designed to permit methane fermentation. To insure adequate algal growth and surface oxygenation in such ponds, it is vital that the waste contain or be provided with adequate nutrients for algae and for methane bacteria. These are normally present in domestic sewage, but certain essential nutrients will be lacking in most agricultural or industrial wastes and, hence, must be added. Typically required additions are carbon, nitrogen, phosphorus, iron, and molybdenum, but it is rare that more than one or two of these substances must be added to attain rapid microbiological growth.

INFECTIOUS AGENTS

Many authorities have reported the fact that sewage bacterial numbers decline in the natural environment including waste ponds. Marais (10) developed theoretical predictions of the numbers of organisms remaining after a specified time and following treatment in a sequence of ponds. There seems little room to doubt that pathogens are at least as sensitive as are coliform bacteria to the inroads of time and temperature on their numbers. Cysts and ova doubtlessly settle rapidly in quiescent ponds, whereas, bacteria which remain suspended are subject to the cyclic changes in pH, temperature, and oxygen levels in the photic zone or the highly reducing environment of the anaerobic zone. McGarry (11) demonstrated that typhoid-like bacteria survived long periods of anaerobiosis, but rapidly disappeared in an aerobic environment. Following an initial two or three-day burst of increase in numbers of coliform bacteria, we found coliform survival in a single unmixed facultative pond to follow the model shown graphically in Figure 1. To use Figure 1, enter the chart with the mean minimum water temperature and proceed parallel to the ordinate to the sloped line indicating an acceptable most probable number of coliform bacteria, then proceed to the left parallel to the absissa and find the required retention period in days. (Example: at 10°C, 83 days are required to reduce an initial MPN of 10^8 per 100 ml to a final MPN of 1000 per 100 ml). It is apparent in comparing Table 1 with Figure 1 that much longer periods of retention are required to attain a low MPN reading than to attain a low BOD level.

In view of the inevitable die-away of coliform bacteria and, hopefully, pathogens, the greatest threat to discharge quality in ponds of adequate detention period is short circuiting. Short circuiting of influent to effluent occurs readily in a single pond due to changes in temperature and wind direction and changes in water strata due to currents and waves. In addition to the selection of an adequate detention period to permit coliform die-away, it is important to design in such a way that short circuiting is minimized. This is done mainly by designing multiple ponds in series and by proper positioning of influent, effluent, and transfer structures.

The influent of a primary pond is generally placed at the center of the pond and near the bottom to permit a 360° contact of raw sewage with the ponds contents and to bring the waste warmth into the fermentation zone. The plume of sewage usually assumes one of the three forms shown in Figure 2.

Transfer structures should withdraw alternately from the center and top of ponds in series. This concept necessitates a minimum number of 4 ponds in series, the initial pond to have a middle discharge, the second pond, a surface discharge, the third pond, a middle discharge, and the final pond, a surface discharge.

In ponds, edge effects are profound. Currents move along the levees, winds slosh waves about, stirring up deposited materials, and on both the leeward and windward sides, upwelling of bottom water rich in nutrients occurs. If possible, therefore, transfers between ponds should be located at right angles to the direction of the prevailing winds, and the intakes and outlets should be located a sufficient distance from the levees to avoid intake of muddy water. With these precautions against short circuiting and adequate detention time, it is often possible to attain discharge MPN levels as low as 10 to 100 in the temperature climates of California. As an alternative, Hom (12) proposed the use of smaller ponds and applying chlorine for disinfection. If land is expensive, the chlorination alternative may be the most economical in spite of the cost of chlorination equipment, chlorine, and higher maintenance costs. However, if the design criteria set forth here are followed, the probability of requiring chlorine to meet a discharge requirement is greatly reduced.

One problem in meeting discharge MPN requirements which cannot be ignored is reinfection of the pond contents with the feces of wild birds. Ponds appear to be warm and attractive to shore and game birds and their droppings can quickly raise the MPN of a pond discharge to unacceptable levels. Deterrents to this problem have ranged from the construction of a shallow (and hence warmer) sacrificial pond (strictly for the birds) to the use of carbide cannons and sustained open hunting season for teen-age nimrods.

The degree to which infectious viral agents of disease are removed in waste stabilization ponds is under investigation at the University of California by Sobsey and Cooper (13). Their results have indicated little if any survival of specifically induced viruses after moderately long detention periods.

PLANT NUTRIENTS

On a long-term basis, nutrients and other materials are always to some extent removed in ponds. If this were not true, waste ponds would grow deeper and larger instead of slowly filling up. In a sense, with respect to nutrients, a pond can be regarded as a controlled eutrophic body through which the potential of a given water to induce eutrophication in downstream bodies is decreased. Ponds can be designed to accomplish predictable degrees of nutrient removal and hence to meet certain quality standards. Except for water, carbon is the most essential plant nutrient, and we have already shown under the BOD section of this paper that a significant quantity of carbon can be eliminated in ponds by way of methane production. With domestic sewage as substrate, about 50 percent of its organic carbon content can be converted to methane; of the remainder, more than half is contained in stable organic compounds, while much of the balance is present as alkalinity and, hence, available for photosynthesis.

If through vertical mixing one prevents the production of methane in ponds, most of the decomposable carbonaceous material is converted to alkalinity and thus becomes available for algal growth. For those unfamiliar with the comcept, the propensity of a given water to support algal growth is termed by us as Algal Growth Potential (AGP). Specifically AGP is defined as the maximum quantity of algae, usually expressed in mg/1 which can be grown in a given aliquot of waste or water when no factor other than its nutrient content limits growth. Specific AGP's can be attributed to each nutrient

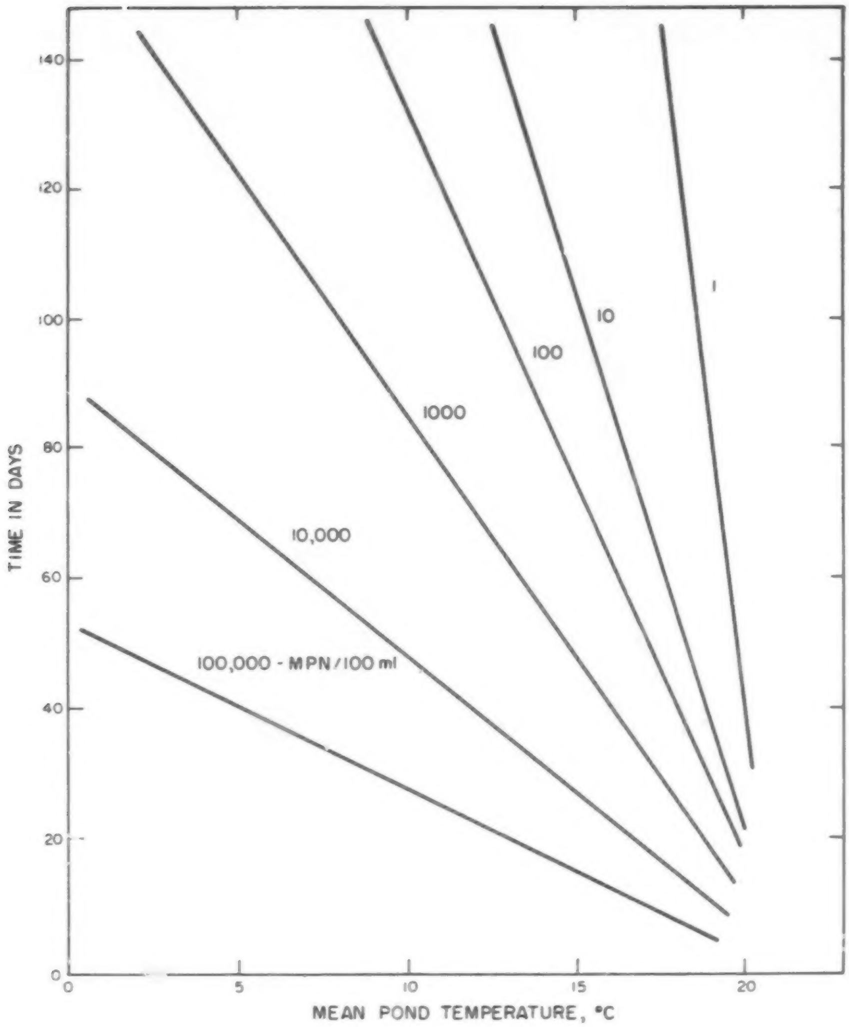

Figure 1. Observed MPN/100 ml in a single facultative pond as a function of time and temperature. Initial MPN 10^8/100 ml

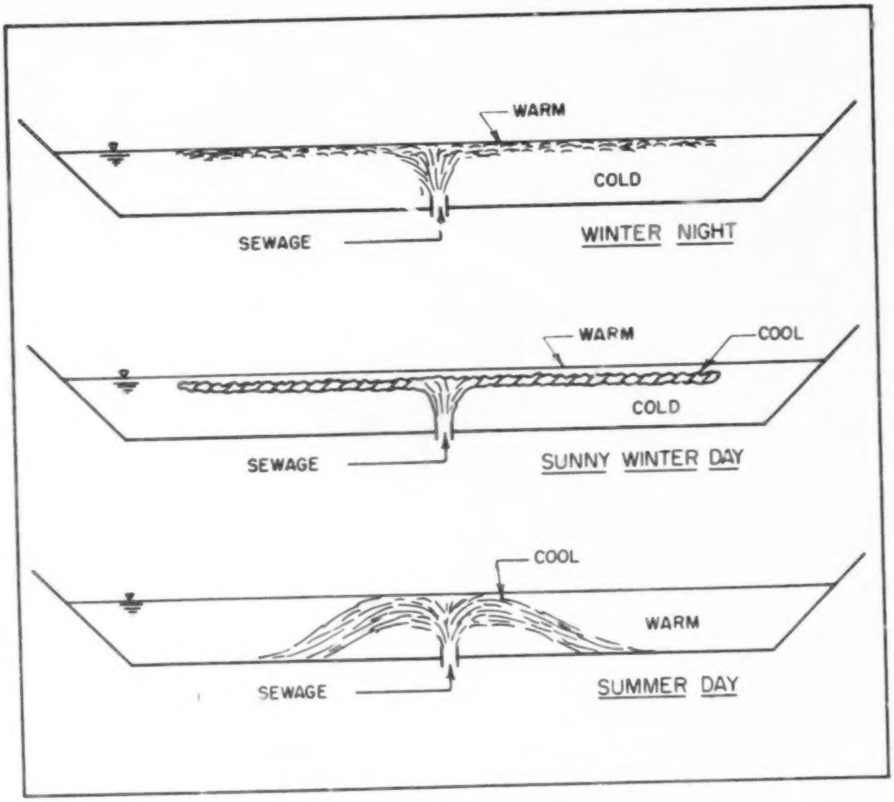

Figure 2. Strata induced in ponds due to thermal differences in sewage and pond water

on the basis of its usual weight fraction in algae. Because sewage is derived from human metabolism which releases carbon dioxide to the atmosphere rather than to the water, and because algae usually contain about 40% carbon on a dry weight basis, the carbonaceous AGP of domestic sewage is only on the order of 300 to 400 mg/1. To attain the concentration of algae necessary to assimilate this much carbon, light must be eliminated as a limiting factor to growth, and shallow reactors must be used. Conversion of AGP to algal biomass thus demands that algal concentration and light limitations be considered in design.

Algae absorb light in the process of photosynthesis and thus influence the depth to which light penetrates algal cultures. The depth of culture in turn limits the attainable concentration of algae. The penetration depth "d" of sunlight into an algal culture may be predicted by Equation 2.

$$d = \frac{\ln S_o^v - S_d^v}{C_e \alpha} \qquad (2)$$

in which S_o^v is the visible light intensity at the surface. S_d^v is the visible light energy intensity at depth d, (usually taken as that light intensity at which the photosynthetic rate of oxygen production is equal to the respiratory rate of oxygen use), C_e is the concentration of uniformly dispersed algae, and α is an absorption coefficient. It is convenient to express S in mg calories/cm^2/min (in full sunlight $S_o^v \cong$ 700 mg calories/cm^2/min), while C_e is usually expressed in mg/1 and d is expressed in cm. A typical value of α in these units is about 1.0×10^{-3} (mg/1 - cm)$^{-1}$.

According to Equation 2, in a completely mixed system, the light limiting concentration of algae is an inverse function of depth. Thus, by assuming a depth and a value of α , S_d^v and S_o^v, we may predict a maximum light limiting uniform concentration of algae which will be attained. For example, if S_d^v is taken as e so that $\log_e S_d^v = 1$ and S_o^v is taken as 700 and $\alpha = 1 \times 10^{-3}$ then $C_e d =$ 5.550. Conversely, if one assumes a concentration of algae equal to the AGP , a specific depth is defined by the same equation.

It thus follows that to incorporate the 300 to 400 mg per liter of carbonaceous AGP of domestic sewage into algal biomass, one would need a continuously stirred reaction pond 5.550/400 = 14 cm deep; similarly, to incorporate a typical 1000 mg per liter nitrogenous AGP of domestic sewage, the pond depth would need to be only 5.55 cm deep, and to exert the typical domestic sewage phosphorus AGP of 2,000 mg/1, a pond only 2.75 cm deep would be required. In the case of exerting the nitrogenous AGP, carbon would have to be added, and if the phosphorus AGP were to be exerted, both carbon and nitrogen would have to be added. Sufficient detention time must always be provided to permit integration of solar energy in the amounts required by algal growth. Essential formulations for this design requirement have been described by Oswald (14) and Shelef (15).

With respect to light, Shelef (15) has shown that the growth rate, μ, is

$$\mu = \mu_{\max} \left[1 - \exp \left(-\frac{I_x}{I_s} \right) \right] \qquad (3)$$

in which μ is the specific growth rate per day, and I_x and I_s are, respectively, light intensity (irradiance) in an incremental depth and light saturation intensity in cal/cm^2/min. The net production rate per unit of illuminated culture surface, P_{na}, is

$$P_{na} = E_m' I_s \left[\alpha_t C_c d + E_i \left(\frac{I_o}{I_s} \right) - E_i \left(\frac{I_d}{I_s} \right) \right] - 10^{-3} K_d C_c d \qquad (4)$$

in which states that the net algal production, P_{na} is a function of the thermodynamic light conversion efficiency, E_m'; the light saturation intensity, I_s, the total extinction coefficient, α_t; the algal concentration, C_c; the culture depth, d; the exponential integral function, E_i, of the ratio between the incident irradiance, I_o, and transmitted light, I_d, and the light saturation intensity I_s. K_d is a decay rate constant to account for losses by endogenous respiration. Shelef has also shown that with respect to nitrogen, the growth rate is described by the following equation:

$$\mu = \mu_{\max} \left[1 - \exp \left(-\frac{S_{N1}}{K_A} \right) \right] \qquad (5)$$

in which S_{N1} is the concentration of nitrogen in the reactor and effluent of continuous flow completely mixed cultures and K_A is the nitrogen concentration at which $\mu = 0.63\ \mu_{\max}$.

Zabat (16) has studied the kinetics of phosphate requirements of *Chlorella pyrenoidosa* (Emerson strain) and found that the growth rate can be expressed as

$$\mu = \mu_{\max} \frac{S_{P_1}}{K_s + S_{P_1}} \qquad (6)$$

where S_{P_1} is the concentration of phosphorus in the reactor and effluent of a chemostatic culture and K_s is the concentration of phosphorus corresponding to $\mu = \frac{\mu_{\max}}{2}$.

The yield of algal cells in terms of wgt. of algae per unit wgt. of phosphorus consumed and the phosphorus content per unit cell mass were found to strongly depend on the dilution rate and phosphorus level in the substrate. Algae yields were on the order of 100 to 350 mg/mg of phosphorus consumed. Higher dilution rates effected lower phosphorus removals than lower dilution rates but favored greater rate of cell production.

Several practical methods are available to overcome the severely shallow pond depth limitations imposed by the high AGP's of domestic sewage. The simplest by far is to apply recirculated effluent from which algal cells and consequently AGP has been removed, thus adjusting the feed AGP to a level desired for a given depth.

Methods of algae harvesting have been extensively explored and described in the literature (17, 18), and hence will not be reviewed here. It should be noted, however, that according to the available evidence, the cost of algae removal from pond effluents in plants on the order of 10 MGD probably will not exceed $20.00 per million gallons. Fortunately, most chemical methods of algae harvesting have a built-in propensity for phosphate removal; thus, recirculation, dilution, and nutrient supplementation need only be applied for carbon and nitrogen removal. The use of recirculation of harvested plant effluent with nutrient supplementation will permit depth adjustment to significantly overcome the problem of mixing in shallow ponds. Once the psychological barrier of algae harvesting is overcome, and the value of ponding as a major beneficial land use becomes recognized, a quantum step will have been made in the field of nutrient recycling and, hence, waste disposal.

ORGANIC CHEMICALS

The problem of hard detergents which caused excessive foaming in the discharge from algal systems can be regarded as solved by the advent of LAS and other soft detergents, but several new problems have arisen. The extensive uses of detergents containing proteolytic enzymes seems to have made algae more subject to preditor attack. It has been established experimentally in Czechoslovakia by Setlik (19) that Papayan Tocadiastase renders *Scenedesmus* more digestible for laboratory rates and mice, the required amount being on the order of 1 mg of enzyme per gram of algae. This is probably the order of magnitude of the current ratio of enzymes to algae in domestic waste ponds, and, hence, the susceptibility of algae to predator attack may be increased. Our experience with the algae production facility at Richmond has been that of increasingly serious attacks of *Scenedesmus* predators, but no definitive work has been undertaken by us in this area.

The affinity which algae have for chlorinated hydrocarbons is well established. In a hitherto unpublished study, we have found that both *Scenedesmus* and *Chlorella* adsorb chlorinated hydrocarbons (CH) in direct proportion to the CH concentrations and that the bond between CH and algae, although strong, is not irreversible. Chlorinated hydrocarbons adsorbed on algae can be removed from algae by repeated washing and centrifuging or by elutriation with oil followed by differential centrifugation. It has thus been concluded by us that the uptake of chlorinated hydrocarbons by algae is adsorptive rather than assimilative. There is, therefore, the definite possibility of utilizing algae in ponds as a method of controlling the concentration of chlorinated hydrocarbons which would otherwise be discharged to the environment. Although costs would be high if this were the sole purpose of the process, removal of CH in a multipurpose project would be more economical.

Pentachlorophenol, a widely used biocide, is extremely toxic to algae. Algal cultures not acclimated to PCP are inhibited when the concentration of pentachlorophenol is as little as 1 part per billion. Through selection, cultures have been developed which can grow in the presence of 1 mg per liter of pentachlorophenol.

In view of the importance of algae to the basic food chain and the increasing use of organic chemicals in the environment, it is remarkable that so little definitive work has been done to study the interaction of specific algal species and specific environmental chemicals. Lewin is currently organizing a symposium as a step in overseeing this deficiency (20).

OTHER MINERALS, CHEMICALS, AND SILTS FROM LAND EROSION

Little has currently been reported regarding the design of ponds to deal with such substances in the environment, but a potential does exist. For example, through modifying the carbonate bicarbonate ratio, algae cause an increase in pH in natural waters and frequently bring about the precipitation of $CaCO_3$, MgOH, and $Ca_3(PO_4)_2$. In studies of algal production in hard water, we found a removal of nearly 50 percent of the calcium and magnesium originally present in the waste water processed (21). Similar results have been observed in the studies of the Interagency Nitrogen Removal Group at Firebaugh, California (22). Water softening by algae increases the sodium ratio of the water and consequently diminishes the value of such water for irrigation. As is well known decrease in hardness enhances the value of water for cooling and for most industrial use. It also renders a water more subject to desalinization and demineralization.

RADIOACTIVE SUBSTANCES

The magnification of radioactive substances in the food chain is well documented and, of course, begins with the uptake of radioactive nuclides by algae. A general rule of thumb is that any polyvalent cation is strongly adsorbed by algae. Thus, the possibility exists of utilizing algae for heavy metal accumulation and control, but this aspect of applied algology has not currently been studied to any significant extent.

HEAT POLLUTION

The use of cooling ponds is a well-established art and one which is likely to see extensive use in the future. If hot wastes are to be excluded from the natural environment, the use of ponds is an obvious alternative to the use of cooling towers or injection wells. The possibility of utilizing ponds as multipurpose waste treatment, recreational, cooling, and food production facilities has not been adequately explored either from a theoretical or practical standpoint. It is interesting to note that only a small fraction of the light energy absorbed by an algal culture is used in photosynthesis, the balance of both the visible and infrared light is converted to heat within the column of water in which it is absorbed. The infrared and visible sunlight combined amount to some 1600 mg cal/cm^2-min and can bring about a very rapid heating of the surface of a body of water containing algae. We have repeatedly observed temperature increases of 10°C. per day in the top 10 cm of completely stratified ponds containing dense algal cultures. We have, in fact, found that a dense algal culture acts as a black body and that per unit of dry weight, living algae have greater light absorption properties than carbon black (23). Dense algal cultures then tend to heat rapidly and stratify by day and cool very rapidly at night thus losing most of the heat gained during the day. However, if the heated daytime surface water is mixed into the main body of water, nocturnal heat loss is diminished. There is little doubt that systems could be strategically designed to take advantage of the unique cooling and heating abilities of ponds containing algae.

FIELD OBSERVATIONS — THE SAINT HELENA WASTE POND SYSTEM

Many of the design concepts set forth above have been embodied in a new advanced ponding system at Saint Helena, California. This system involves decarbonation and denitrification in a 10-foot deep facultative pond (pond No. 1) followed by algal growth in a high-rate pond (pond No. 2) with algal sedimentation in a deep pond (pond No. 3) and effluent clarification in ponds No. 4 and 5. A schematic diagram of this system is shown in Figure 3 and design characteristics are given in Table 3. Performance characteristics are tabulated in Table 4. It is evident from the data that water quality control of a high order is attained in the Saint Helena waste pond system and that this control is attained through application of the design principles set forth in this paper. Although algae removal is unnecessary in the Saint Helena ponds because they have no dry weather discharge, should algae harvesting be included as an added feature, effluent quality will be further improved particularly with respect to phosphorus.

Based on nearly three years of operating experience, we have concluded that although unforeseen problems may arise at Saint Helena, the current evidence is that a rational design has been accomplished which meets or exceeds all water quality discharge requirements for that environment.

Table 3 – Capacities and Detention Times of the Ponds

Pond	Area (acres)	Liquid Depth (feet)	Volume (ac-feet)	Detention Time (days)
Facultative	2.2	10	22.0	17.1
Aeration	5.0	3	15.0	11.7
Algae Sedimentation	2.1	8	16.8	13.1
Maturation I	4.3	8	34.4	26.8
Maturation II	5.9	9	53.1	41.3
TOTAL	19.5		141.3	110.0

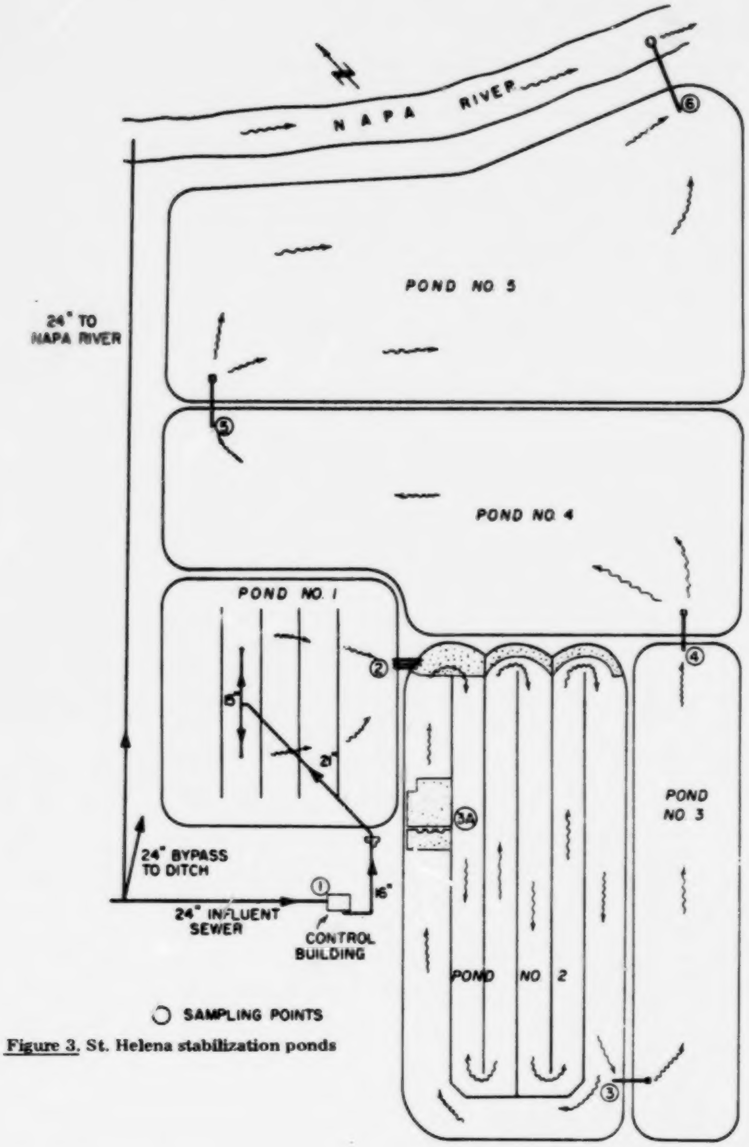
NAPA RIVER
POND NO. 5
24" TO NAPA RIVER
POND NO. 4
POND NO. 1
POND NO. 3
POND NO. 2
24" BYPASS TO DITCH
24" INFLUENT SEWER
CONTROL BUILDING
16"
15"
21"
①
②
③
3A
④
⑤
⑥
○ SAMPLING POINTS

Figure 3. St. Helena stabilization ponds

Table 4 – Performance of the Saint Helena Ponding System

Characteristic	Sampling points						% Removal
	1	2	3	4	5	6	
BOD	142.5	22.2	8.2	5.1	2.1	3.9	97.2
COD	307.0	115.8	60.9	39.0	22.7	2[illegible].6	92.8
Carbon (C)	131.5	63.4	51.0	42.6	34.8	28.0	78.8
Nitrogen (N)	35.4	11.5	9.5	6.6	5.0	3.1	91.6
Phosphate (P)	11.8	7.5	6.7	5.4	4.4	4.2	64.3

Values in mg/l
1. Influent
2. Facultative Pond Effl.
3. Aeration Pond Effl.
4. Algae Sedimentation Pond Effl.
5. 1st Maturation Pond Effl.
6. 2nd Maturation Pond Effl.

ACKNOWLEDGMENTS

This study was supported in part by Research Grant WP 00975-03 from the Federal Water Quality Administration and by supplementary funds from the University of California and the Rockefeller Foundation. We are indebted to Mr. Gordon Barber of the Saint Helena treatment plant and to Mr. John Stauff of Yoder, Trotter, Orlob Associates, Walnut Creek, California, engineer of the plant. Their cooperation greatly facilitated our work. We are also indebted to Mr. Henry Gee, Research Specialist of the University for conducting much of the analytical work and for preparing the figures.

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CURRENT DESIGN CRITERIA FOR OXIDATION PONDS

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INTRODUCTION

The waste oxidation pond has become a "standard" for many consulting engineers to use in developing a waste treatment facility for small-to medium-size communities. The purpose of the paper is to report current design criteria in use by various State water pollution control agencies, to explain some of the operating experiences that have influenced design considerations, and to suggest areas that should be considered for further evaluation. Our subject is the conventional process without supplemental aeration or design for deliberate anaerobic conditions.

Design criteria for oxidation ponds have evolved from

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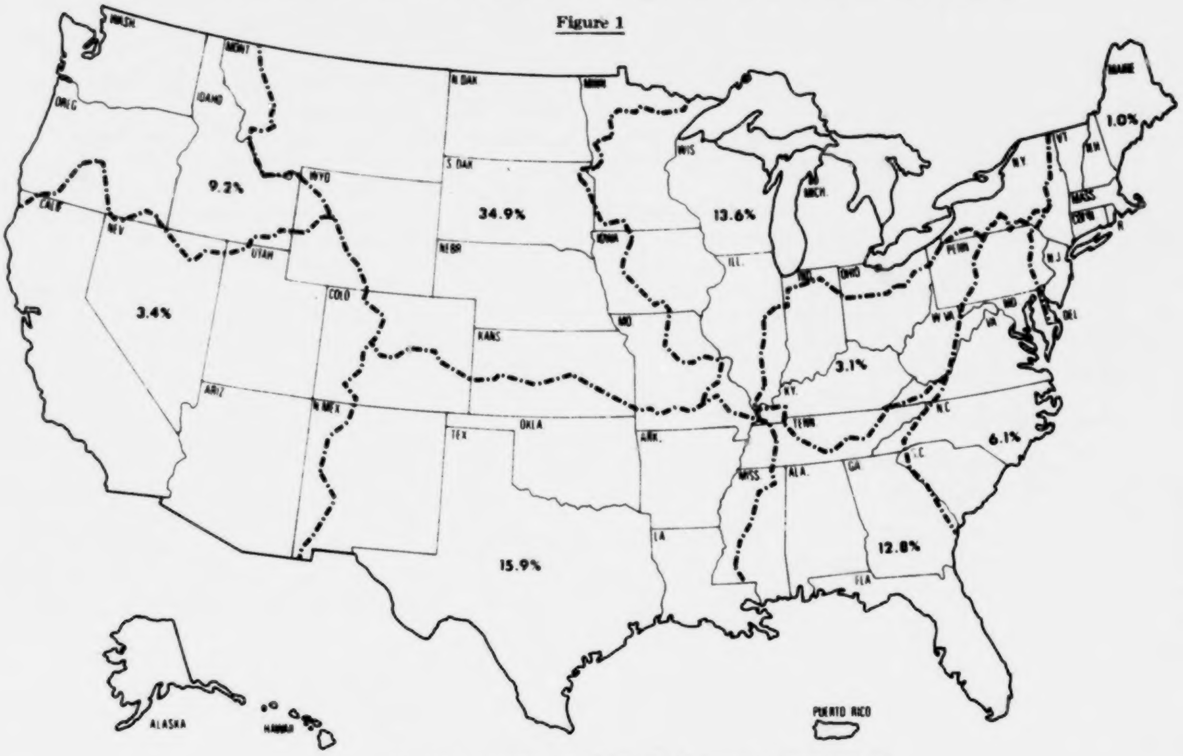
FEDERAL WATER QUALITY ADMINISTRATION REGIONS

Figure 1



Regional Location of Ponds by Percent of Plant Projects (as of September 30, 1967)

practical application using the "cut and try" approach rather than the scientific approach used in development of atomic devices and for space exploration. The research community has had only casual interest in the evaluation of performance of the lowly waste stabilization pond. Lack of interest has been primarily due to the small size of most communities served by ponds.

A study of all treatment facilities built in the period 1956 to 1967 under the Construction Grants Program of the Federal Water Pollution Control Act indicated that approximately one-third of 6,000 projects were waste oxidation ponds (1). Figure 1 shows by Regions of the Federal Water Quality Administration the percentage of all ponds constructed in the U. S. with grant funds during that period. These percentages clearly reflect primary use in less urbanized areas with proper climatic conditions.

The size of communities and waste load is small since this device is generally best suited for the small- to medium-size community. There are exceptions to this practice; however, the predominate use has been in rural areas for small communities where the sun shines and the winds blow a major portion of the year.

Basic design criteria originally came from accidents where raw discharges of municipal wastes were impounded by a natural formation with the unexpected results of retention of sewage without the development of nuisance conditions.

DESIGN PARAMETERS

States without formal criteria but which will approve the process are: Alaska, California, Delaware, Hawaii, Nevada, New Jersey, and New Mexico. Certain States, Florida and Texas, have criteria for waste stabilization ponds following other forms of treatment. Connecticut, Massachusetts, Rhode Island, and Vermont do not endorse their use. The Missouri Basin Engineering Health Council adopted a committee report on "Design, Construction, and Operation Practices among Missouri Basin States" on January 21, 1960 (2). The Great Lakes-Upper Mississippi Board of State Sanitary Engineers included guidelines based on their experiences and needs in "Recommended Standard for Sewage Works (Ten State Standards)" adopted on May 10, 1960. These two documents are the major design guidelines available for use by State water pollution control agencies and consulting engineers. The latter document was again revised in 1967 (3).

"Ten State Standards" is the design criteria in Iowa, Indiana, Kentucky, Louisiana, Massachusetts, Minnesota, New York, Rhode Island, and Wisconsin. Michigan follows the "Ten State Standards" closely. Land area acquisition for long term growth is required, but actual lagoon construction for short term growth is permitted. A detention period equivalent to six months flow is required since discharges are controlled and generally allowed only twice per year (4).

Mississippi will allow the use of a single cell for a design population equivalent of 2,000 and from 2,000 to 5,500 a three cell pond. A general requirement is that any treatment works must remove 85 percent of the BOD and suspended solids.

Design parameters have been divided into two divisions: Table 1, Basis of Design; and Table 2, Layout and Construction Details. These tables do not reflect the less stringent requirements sometimes specified for small installations.

The "Ten State Standards" are the basis for Table 2, and numbers shown at the top of the table refer to the same paragraph number in the reference. Where a number is shown, an agency has adopted a different or more stringent requirement which must be substituted for the requirement in the reference. Table 1 must be used as the basis for specific depth requirements in Item 4.42, Table 2.

VARIATIONS IN DESIGN PARAMETERS

Primary cell loadings for each State have been shown in Figure 2. January solar radiation was chosen as reflecting minimum radiation conditions and thus limiting the photosynthetic rate. The time of 180 days was taken as a measure of the ice free period. At present, experience is the best guide to a loading rate which will balance solar radiation, ice cover, reliability of effluent quality, possibility of odor production, and the host of other factors that make a successful facility.

Detention time, which is related to loading, varies from 20 to 180 days. Longer detention times are specified where winter flows must be held due to pond effluent quality during this period.

Most loading problems have arisen over inadequate evaluation of industrial waste discharge. Whey, blood, or similar byproducts go a long way towards creating odor and poor effluent production.

The trend to multiple cells is obvious. In the Southeast, criteria are being revised to reflect the results of research in this area. The reduction allowed in the primary cell is still evolving and will probably vary with location and mode of operation.

There is no major disagreement over either minimum of maximum depths. The minimum depth has been chosen to suppress willow and cattail growth.

Items other than those shown in Tables 1 and 2 in State criteria reflect different approaches to various individual problems. These items have been grouped for discussion purposes.

An all-weather access road is specifically required in Arizona, Georgia, Missouri, Nebraska, and Oklahoma. Access to any treatment process should be provided so that operation and maintenance can be performed.

Two States, Georgia and Oregon, set a lower limit of one-half acre on cell sizes while Montana has an upper limit of 40 acres. The minimum reflects a size to achieve mixing. Velocity control to reduce erosion is the basis for the upper limit.

A number of States have set definite limits on seepage, Table 2, but Alabama's and Missouri's approach requires an on-site investigation by the Geological Survey, while Nebraska requires a laboratory analysis of the soil to determine permeability. Proof of satisfactory rate of seepage can be found by prefilling, which is required in Colorada, Georgia, Nebraska, Oregon, and Tennessee. Kansas and Colorado require that seepage plus maximum evaporation must not exceed the inflow so that a minimum water leval of 1.5 feet and 3.0 feet respectively can be maintained at all times.

The State of Georgia requires a lean Portland cement pavement on the slope of dikes with a width six inches above the high water level and 18 inches below the low water level to prevent erosion and weed growth. Nebraska requires rip-rap extending two feet above and two feet below maximum and minimum water level. Kansas requires soil sterilization below the maximum water line. Low rainfall in Utah reverses the problem of growth and a method other than seeding is needed for dike stabilization.

Alabama, Arizona, and Georgia do not recommend that the inlet pipe be brought in on zero grade. Arkansas and Ohio will allow the inlet pipe to be placed on the bottom. Wyoming will allow the inlet pipe to be placed on the bottom or on posts. Ice, grit, and other factors influence this choice, but where feasible, bringing the line in on piers seems to be acceptable.

Arkansas, Georgia, Idaho, Montana, and Tennessee have provided that the point of influent discharge needs to be at the third point, at the one-third point on small ponds, and at the one-fourth point on larger ponds, 200 to 400 feet from the dike, no greater than 400 feet from the dike, and one-third the distance across the pond, respectively. Tennessee criteria are used in conjunction with a recommended layout of either square or circular ponds. Alabama required multiple inlets for single ponds of 10 acres in area or greater.

In Idaho and Montana, ponds must be designed to permit lowering liquid levels at a rate of one foot per week, while the normal load is being received.

New Hampshire requires the use of either metal or fiberglass in outlet structures. Alabama does not require the lowest outlet to be 12 inches off the bottom. Generally the use of a baffle, which is implied in the "Ten State Standards" reference, is specified in most States' criteria.

Disinfection is required by Idaho, Oregon, New Hampshire, and Washington. Utah requires chlorination, unless more than three cells are provided, and the discharge rate is no more than twice the inflow at maximum depth. Georgia requires multiple cells with 90-days detention where bacterial reductions are critical.

OPERATIONAL EXPERIENCES

We will not repeat the history of lagoons because that has been adequately documented by others (5). We would like to report a few instances of design criteria developments that were personal experiences of the authors or were reported by others. These serve to illustrate the general type of experience that has dictated changes in design. For example, most States in the Missouri Basin started with the conservative design loading of 20 to 25 pound of five-day BOD per acre per day. Because of extreme winter temperatures, the primary design consideration for the Northern States has been retention time. Consequently, it became necessary to maintain complete retention of all wastes during the winter months. Other States have been able to increase loading much higher, and obtain satisfactory results.

TABLE I

BASIS OF DESIGN

	Pre-treat-ment Req/d	Loading (#/Acre/Day)	Detention (Days)	Primary Pond Reduc.	Depth (Feet)	Multiple Units Req/d
ALABAMA	No	50	30	--	3 - 5	No
ARKANSAS	No	30	--	--	3 - 5	No
ARIZONA	Above 2500 PE	50	20	--	3 - 5	No
COLORADO	No	21.8	30	--	0 - 5	Above 10 Acres
FLORIDA	Yes	12½ - 50	--	--	3 - 5	No
GEORGIA	No	35	--	--	3 - 5	When loading under 25 lbs.
IDAHO	No	22 - 34	60	--	3 - 5	2
ILLINOIS	No	22 - 30	--	75	3 - 5	W.R.
KANSAS	No	35	90	--	3 - 5	3
MAINE	No	20 - 35	--	--	2 - 5	Yes
MARYLAND	No	35	90 - 100	--	2 - 5	W.R.
MISSOURI	No	45	--	70	2 - 5	2
MONTANA	No	18 - 20	Winter	--	2 - 5	2
NEBRASKA	No	25	--	--	2 - 5	Above 2 Acres
NEW HAMPSHIRE	Comminutor	35	--	--	2 - 5	2 - 4
NORTH CAROLINA	W.R.	50	30	--	3 - 5	W.R.
NORTH DAKOTA	No	30	150	70	2 - 5	2
OHIO	No	34	--	--	2 - 5	2
OKLAHOMA	No	30	--	--	3 - 5	2
OREGON	No	35	--	Overall Loading not Greater than 35 lbs	3 - 5	2
PENNSYLVANIA	No	22	--	75	2 - 5	For Secondary
SOUTH CAROLINA	No	50	30	85	3 - 5	W.R.
SOUTH DAKOTA	No	17	Winter	--	2 - 5	2
TENNESSEE	No	34	--	75	4 - 5	2
TEXAS	Yes	75	--	Overall Loading not Greater than 35 lbs	3 - 5	2
UTAH	No	40	60 - 120	33	3 - 6	3
VERMONT	No	20	--	--	3 - 5	W.R.
VIRGINIA	No	80	25	75	3 - 5	W.R.
WASHINGTON	No	40 - 50	Winter	--	3 - 5	--
W. VIRGINIA	No	34	--	--	3 - 5	--
WYOMING	No	20	--	--	2 - 5	--

Loading based on 0.17 lbs/day/person
W.R. - When required

TABLE II

LAYOUT AND CONSTRUCTION DETAILS

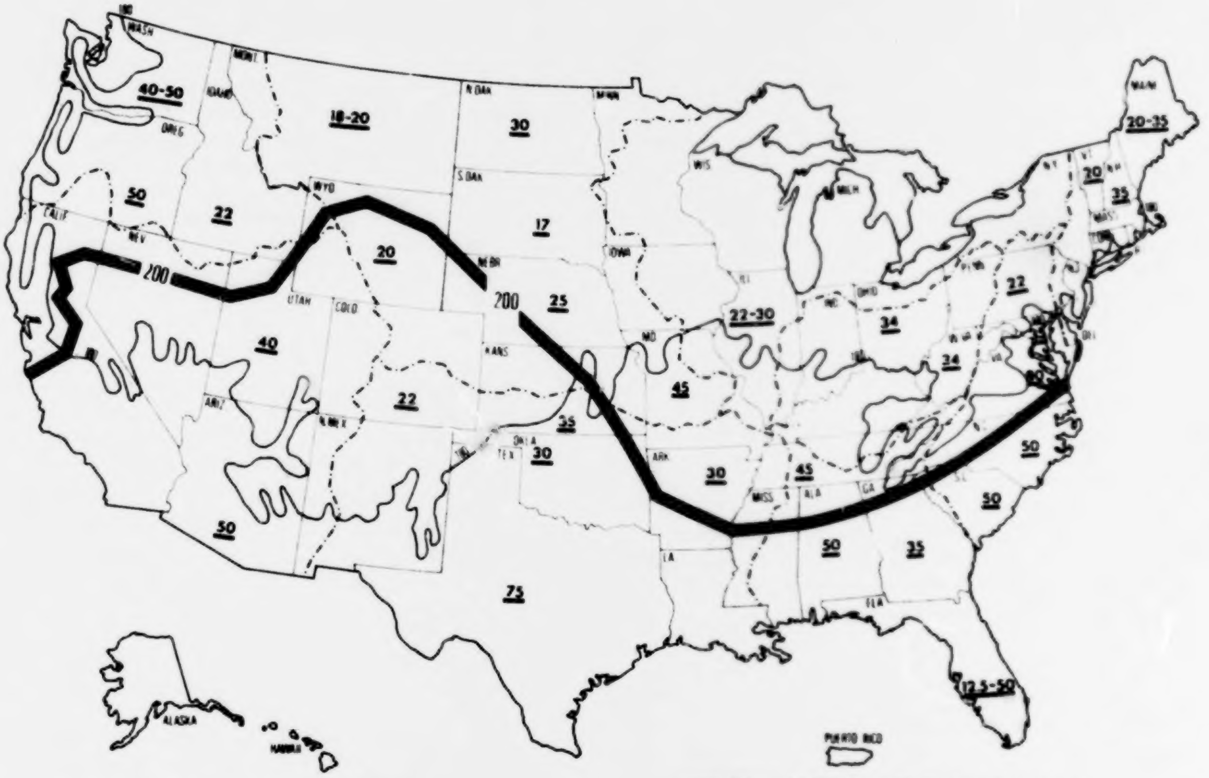
	1.11	1.12	1.13	1.14	1.15	2.5	3.1	3.2	3.3	3.4	4.11	4.12	4.131	4.132	4.141	4.15	4.18	4.19	4.21	4.22	4.23	4.24	4.31	4.32	4.33	4.34	4.35	4.36	4.41	4.42	4.43	5.1	5.2	5.3	5.4
ALABAMA		X				X	X		X	X	X	X	X			2	X		X		X					X		3		X		X	X		
ARIZONA	X	X	X	X				X			X		X	X		X			X	X			X			X		X		X		X	X	X	
ARKANSAS							1300	X		X	X	X	X	X		2	X		X			X	X					X	X	X	X	X	X		
COLORADO	X	X	X	X		X					X	X	X	X	6	2	X		X	X		X	X	X	X	X		X	X	X	X	X	X	X	
GEORGIA		X				X	300	X	X	X	X	X	X	X	X	X	X		X	X	X	X	X	X		X		15			X	X	X	X	
IDAHO						X	X	X	X	X	X	X	X	X	5	X	X	X	6	X	X	X	X	X	X		X	X		X	X	X	X	X	X
KANSAS												X	X	X		X	X					X									X				
MAINE	X	X	X	X	X	X	1500	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
MISSOURI	1300		X	X	X	X	X	X	X	X	X	X	X	X	X	2	X	X	X	X	X	X	X	X	X	X	X	3		X		X	X	X	
MONTANA						X	X	X	X	X	X	X	[illegible]	2	X	X	X		X	X	X	¼	X		X	X	X	X		X	X	X	X	X	
NEBRASKA							1300	X	X	X	X	X	X	X		X	X	X		X		¼				X		4		X	X	X		X	
NEW HAMPSHIRE							6000	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	
NORTH DAKOTA							X		X	X	X	X	X	X	6		X		X	X										X	X	X	X		
OHIO					X	X	1000	X	X		X	X	X	X		X	X		X			X								X	X	X	X	X	
OKLAHOMA						X		X	X	X	X	X	X	X		X	[illegible]		X	X	X	X		X		X	X	X		X		X	X	X	
OREGON						X	1300		XX	X	X	X	X	X	X	X	X		X	X		¼				X				X		X	X	X	
SOUTH DAKOTA						X	X	X	X	X	X	X	X	X	5	X	X		X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	
TENNESSEE							1000		X		X	10	X	X		X	X		X	X	X	X								X		X	X		
TEXAS	X	X	X			X			X		X	X	4			X	X			X	X	X				X				X	X				
UTAH	X	X	X			X	1000	X	X	X	X	X	X	X	X	X		X	X	X	X	¼	X	X	X	X	X	X		X	X	X	X	X	
VERMONT		X				X	1300	X				6	X	X	6	X	X		X	X	X		X			X				X		X	X		
WASHINGTON				X		X	X	X	X	X	X	X	X		X	X	X	X	X	X	X	¼	X	X	X	X	X	3	X	X	X	X	X	X	X
WYOMING						X	1300	X	X	X	X	X	X	X		X	X		X	X						X				X+	X	X	X		
WEST VIRGINIA						X	1300	X	X	X	X	X	X	X	6	X	X		X	X	X				X			X	X			X	X		

UNITS:

Item 3.1 - Ft.
Item 4.24 - "/Day
Item 5.3 - 0; outlet

FEDERAL WATER QUALITY ADMINISTRATION

Figure 2


WASH
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50
75
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22-30
34
24
22
50
50
45
30
50
35
20-35
20
35
12.5-50
ALASKA
HAWAII
PUERTO RICO

— Pond Loading (lbs. BOD$_5$/acre/day)

— Mean Daily Solar Radiation (Langleys) January

— Mean Length of Freeze-Free Period (Days)

Between Last 32° (F) Temperature in Spring and First 32° (F) Temperature in

It was found that many small, single cell lagoons were not filling sufficiently during the first year to prevent emergent vegetation from becoming a nuisance and a potential health problem through creation of mosquito breeding areas. The consensus of opinion was that multiple cell lagoons should be required for all installations and one cell ponds approved only where the load was so small that it would not be practical to build two cells. South Dakota and Nebraska have permitted a two level bottom on a single cell lagoon with the lower level two to three feet below the upper bench. This reduced the area for initial filling and an adequate water level could be maintained during the period of initial hook-up and low water use for a new system. Additionally, the problems with the "start up" period for ponds dictated a design requirement for either soil sterilization or prefilling to prevent vegetation from becoming a permanent problem in the ponds.

A rational evaluation of initial, as well as design flow and net water losses, would have prevented many of these problems. With a seepage rate of one-fourth inch per day, net evaporation of 0.10 inch per day, and an actual loading of 100 persons per acre per day, the per capita flow must be 95 gallons to balance losses. The use of a rate of 100 gpcd, which is standard, unfortunately is not realistic in small communities.

The authors are not recommending a general seepage rate, which only an on-site geological evaluation or soil analysis will supply. Too often possible pollution of the ground water resource has been substituted for surface water pollution.

The practice of providing sealers or linings has developed from the need to repair conventional lagoons that have developed leakage problems. Some of the following materials and methods have been utilized for sealing:

1. Bentonite Clay
2. Chemical
3. Soil Cement
4. Clay Barrow
5. Asphalt Liner
6. Plastic Liner
7. Lean Portland Cement

None of these materials are suitable for all situations; each should be evaluated for the location and problems at hand.

Another significant design consideration that was found by the "mistake" type of research was the need for bank protection for larger lagoons which were subject to more or less constant wave action. This led to a routine question by the regulatory agency concerning the consulting engineer's evaluation of the need for bank protection. The use of flatter slopes of six or seven to one has been tried, but without reported real success. Wave action was found to still cause erosion.

In the more arid parts of the country, the ultimate treatment is provided in many cases with oxidation ponds through complete retention of all wastes where the inflow is balanced by evaporation. In these cases, more cells are required to maintain a water level in at least one of the cells at all times. The State water pollution control agency will sometimes require a well to be constructed at the site to insure a supplemental source of water.

The use of series-operated multi-cell oxidation ponds, while initiated for other considerations, is probably one of the most important design changes. Many problems found with oxidation ponds can be eliminated or greatly reduced by use of multiple cells. For example, series operated cells show a marked reduction of coliform in the effluent.

In situations where a minimum of 90 percent organic removal is required vector control is a constant problem, ground water is a source of water supply, and the area is rapidly urbanizing, the use of oxidation ponds is not prudent. This is the situation in Florida.

SUMMARY

In the ten years that have elapsed since the last Symposium, the profession has experienced how to keep sewage in and people out of ponds. While this device has served well in the past, times have changed. The State and Federal stream standards must now be met. The nature and use of the receiving stream must be considered for process selection; for example, the density of fecal coliform bacteria in single cell pond effluent cannot be ignored. The future demands that removal of algae must receive more attention. This week we hope to learn what others are doing to define effluent characteristics and the causes of variations in the effluent.

Results must be based on standard analytical methods for defining effluent characteristics, and these do not now exist. One is never quite sure in reviewing the work of others whether the analytical methods used include the algae in the effluent. Variations in effluent quality must be related to improved methods of design and operation to cause a high quality effluent. Since basic hydro-dynamics and biochemical transformations are difficult to understand, the task of standardization is not simple.

Where conditions are right, the oxidation pond will provide the most economical solution to waste treatment both on capital and operating costs. This does not mean that the pond is the solution for every situation, but rather that the designer must use his ingenuity to economically meet the needs of his client while protecting our water resources.

ACKNOWLEDGEMENT

The State design criteria for oxidation ponds were initially supplied by Federal Water Quality Administration's Regional Offices. After a review and compilation of the criteria had been made each State was recontacted for verification of the material. The only exception was in the Northeast Region as the Region had used a questionnaire to obtain design data from each State. Confirming replies were received from all but three of the remaining States.

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NEW EXPERIMENTAL POND DATA

E. F. Gloyna[2]
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INTRODUCTION

Years of field practice have established the waste stabilization ponds as a low cost and reliable method of treatment. The ability of waste stabilization pond systems to reduce the biochemical oxygen demand (BOD) is well known and mathematical models have been developed to represent the kinetics of waste stabilization in ponds. However, owing to the complexity and interdependency of the reaction taking place within a pond, general design criteria have been difficult to develop.

One of the earlier designs dealing with waste stabilization ponds was based on a detention period of 200 days or 10 acres of lagoon per 1000 people as suggested by VanHeuvelen and Svore (1954). Within a few years, the design of waste stabilization ponds was handled primarily on areal loading. Pond loading varied according to the geographical location, but loading of 20 to 70 lbs BOD_5/acre/day were common. The lower loadings were used primarily in areas where ice cover occurred during the winter. The higher loadings were used in the southern states. An average loading of about 50 lbs BOD_5/acre/day became the accepted practice.

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During the period encompassing the early 1950's to the early 1960's, several investigations were in the process of developing rational design formulations applicable to general pond design. Hermann and Gloyna (1958) developed a design relationship based on 90% removal of organic material, which took into consideration such factors as temperature and detention time. Oswald (1960) oriented his waste stabilization pond design toward a maximum utilization of available light energy for maximum algae production, and Marais (1966) formulated waste stabilization pond design criteria based on the completely mixed system concept.

Recent studies have pointed out the importance of anaerobic fermentation in waste stabilization ponds. Oswald (1963) was one of the first to recognize the importance of the anaerobic sludge zone in the behavior of ponds. He showed that the intensity of anaerobic fermentation in the sludge is greatly influenced by temperature. A rise in temperature of 5 degrees centigrade resulting in approximately a sevenfold increase of gas evolution within the temperature range was experienced. VanEck and Simpson (1966) found the composition of the gas produced in anaerobic ponds to be mostly methane (70 to 80%) and carbon dioxide (10 to 20%) with small amounts of nitrogen (4 to 8%) present. Oswald (1963) found a similar composition of gas produced from the anaerobic sludge layer of a facultative pond. Hence, the gaseous end products of anaerobic fermentation, in particular methane fermentation, are major sinks of carbon from ponds.

Dealing in particular with facultative waste stabilization ponds, the algae-bacteria symbiosis does not result in an acceptable reduction of carbonaceous material if the algae are included in the analysis. In the aerobic surface layer provided by the algae, bacterial oxidation converts the available organic material to carbon dioxide and new cells. The algae incorporate the carbon dioxide into their cell protoplasm and the net result is that practically all of the carbon lost from the aerobic layer is that which goes out in the effluent as algal cells. However, a large portion (40 to 60% as BOD) of the organic material in domestic wastes is in the form of settleable solids. These solids settle out and form a sludge blanket over the entire bottom of facultative waste stabilization ponds. Anaerobic conditions develop within the sludge and, after an acclimation period allows the fermenting organisms to become established, an equilibrium is reached where the amount of fresh sludge being deposited is nearly equal to the amount of sludge reduced to fermentation products.

Anaerobic fermentation is a two step process extremely sensitive to environmental conditions. Anaerobic fermentation is a result of the action of two different types of bacteria: the acid forming bacteria and the methane producing bacteria. The acid forming bacteria take the complex organic matter in the sludge and convert it to simple organic acids, carbon dioxide and ammonia. The methane bacteria then take the organic acids and transform them into methane, carbon dioxide, hydrogen and water. Methane fermentation, the rate limiting reaction, is characterized by a narrow pH range (6.8 to 7.2), temperature in excess of 15°C and strict anaerobic conditions. Hence, methane fermentation is highly sensitive to changes in environmental conditions.

Oswald (1968) has proposed the use of specially constructed sludge deposition zones within primary facultative waste stabilization in order to enhance conditions favorable to methane fermentation within the sludge layer. He felt that by providing a compact sludge deposition zone within a larger pond favorable conditions were more likely to prevail throughout the seasonal variations that the rest of the pond would experience. However, it has been suggested (McGarry and Pescod, 1970) that a higher rate of gas evolution will result from a larger surface area of sludge blanket exposed to the overlying liquid.

In view of the aforementioned difficulties, it was felt that further research into the desposition of the anaerobic sludge fermentation zone would yield valuable information for the formulation of improved waste stabilization pond design criteria. Hence, the primary objective of the investigations reported herein was to develop improved waste stabilization pond criteria from the comparison in behavior and performance of pond systems differ by the location of the anaerobic sludge deposition zone. Parallel studies were also conducted in the laboratory to compare the performance of lab-scale with pilot-scale units.

An experimental facility was constructed at the Govalle Sewage Treatment Plant of the City of Austin, Texas. The experimental facility consisted of three sets of laboratory and pilot-scale pond systems. Each set of ponds was designed to provide uniform control of surface area and volume. The major variable was the location of the anaerobic sludge deposition zone. Its influence on BOD reduction, effluent appearance, general odors and bacterial reduction were evaluated. The three pond systems were similar in all respects except for the placement of the anaerobic sludge deposition zone. System I incorporates an anaerobic pretreatment pond that discharges into a larger facultative pond. System II provides for sludge accumulation in a small trench within the facultative pond, and System III consists of a conventional facultative pond used as a control. All three systems are followed by maturation ponds to improve the bacterial quality of the effluent. A schematic diagram of the three ponds systems is shown in Figure 1 and the physical and operational characteristics of the pilot-scale ponds are presented in Table 1.

In the evaluation of the performance of waste stabilization ponds, in treating domestic sewage, three areas were considered to be of vital importance. These three vital areas were: 1) removal of organic material, 2) removal of suspended material, and 3) reduction of bacterial numbers. This paper will be concerned primarily with the first two items listed above since the bacterial reduction figures have been reported elsewhere (Davis and Gloyna, 1970).

The influence of the location of the anaerobic sludge deposition zone, in the different experimental units, was determined by comparison of the organic removal efficiencies as measured by BOD_5, COD and TOC. The removal of suspended material was determined from suspended solids analysis performed on membrane filters. General pond characteristics were evaluated through measurement of such parameters as: pH, DO, temperature, nutrients (nitrogen and phosphorus) and algal typing.

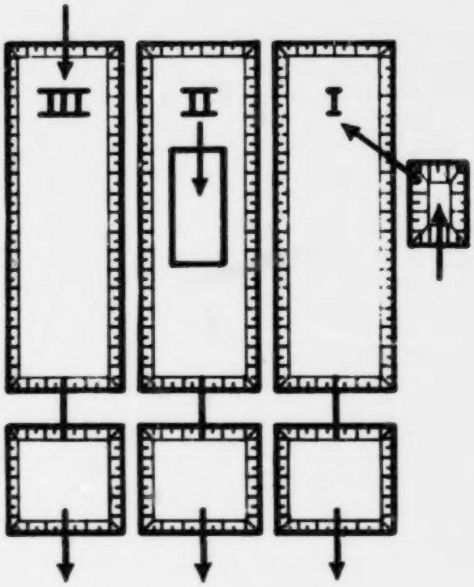

Figure 1. Pilot-Scale Ponds

Table 1. Physical and Operational Characteristics of the Pilot-Scale Pond Systems

Pond System	Depth (ft)	Surface Area (ft^2)	Volume (ft^3)	Flow Rate		Detention (days)	Surface Loading (lbs BOD$_u$/ac-d)	Volumetric Loading (lbs BOD$_u$/1000 ft^3-d)
				(gpm)	(ft^3/d)			
I Anaerobic	10	2,760	8,970	10	1925	4.7	642	4.7
				20	3850	2.3	1284	9.4
Facultative	5.5	25,800	117,530	10	1925	61.0	46	0.23
				20	3850	30.5	92	0.47
II Facultative (w/Trench)	5.5	25,800	126,300	10	1925	65.6	72	0.34
				20	3850	32.8	144	0.68
III Facultative	6	25,800	126,300	10	1925	65.6	72	0.34
				20	3850	32.8	144	0.68
I, II, & III Maturation	4	6,000	18,130	10	1925	9.4	41	0.31
				20	3850	4.7	82	0.62

RESULTS

The experimental lab and pilot-scale ponds, consisting of three separate systems each operating in parallel, were designed to attain a BOD removal efficiency in excess of 90%. Required pond volumes were determined from the Hermann and Gloyna (1958) relationship, which fixed the flow rate to the pilot-scale pond systems at 10 gpm each or 14,500 gpd. Based on an average value of the Austin sewage (350 mg/l BOD ultimate), it was estimated that the primary facultative ponds (Systems II and III) would receive an organic load of about 70 lbs BOD$_u$/acre/day at a detention period of 65 days.

The lab-scale units, set up to compare their performance with respect to the larger pilot-scale ponds, were scaled down hydraulically to operate at a flow rate of 1 liter per day and a detention period of 90 days. However, owing to the dimensions of the aquaria used as laboratory units, the organic load was estimated at about 10 lbs BOD$_u$/acre/day, which was considerably less than that applied to the pilot-scale systems.

Pilot-Scale Pond Systems

The three pilot-scale waste stabilization pond systems were constructed on an unoccupied area of the Govalle Sewage Treatment Plant. Due to the porous nature of the soil in this area it was decided to have the ponds completely surfaced with a plastic liner 10 mils thick. After the ponds were filled with secondary effluent from the sewage treatment plant, a healthy algal growth was allowed to develop before the ponds were finally started up on a continuous basis in June of 1969. Pumps with a maximum flow rate capacity of 50 gpm were used to feed each pond system.

The pond systems were operated for eight months (June 1969 to January 1970) at the design flow rate of 14,500 gpd. During the latter part of February 1970, the flow rate to the pond systems was doubled to approximately 29,000 gpd. The decision to increase the flow rate was taken since pond performance was remarkably good during the winter months of December and January. The following presentation of the data is divided up into two sections; each section describing pond performance during each of the separate loading conditions.

Initial Loading at 70 lbs BOD$_u$/acre/day

Within the eight months duration of this period the pond systems experienced both extremes of weather conditions occurring in the Austin, Texas area. Mean monthly temperature values measured at the Austin Weather Bureau are presented in Table 2. Temperature ranged from a mean monthly low of 7°C in January 1970 to a high of 30°C in July 1969. Water temperatures were generally several degrees higher than ambient temperatures. However, these were obtained from grab samples taken at the pond surface and, since pond temperatures varied considerably with depth, it was felt that water temperatures would not be of very much value for design considerations.

Mean monthly pH and suspended solids data are summarized in Table 3. Although a definite trend was not established, minimum values during the warmer season. The pH data does not reveal any marked differences between the three pond systems, but the suspended solids show a substantial difference between System I and Systems II and III. Waste Stabilization Pond System I, was the anaerobic pretreatment pond, exhibits a noticeably lower suspended solids concentration than the other two systems. The anaerobic pretreatment pond, at a loading of about 640 lbs BOD$_u$/acre/day, was able to reduce the load on the facultative pond to the point where, limited bacterial oxidation in the secondary pond failed to produce sufficient carbon dioxide to maintain as large an algal population as found in the primary facultative ponds. In terms of effluent quality criteria, lower suspended solids would be one advantage of using an anaerobic pretreatment pond in series with a facultative pond.

Occasional dissolved oxygen measurements revealed an aerobic zone, of up to 10 mg/l DO from 6 to 12 inches deep, on the surface of the anaerobic pond during the entire summer season and most of the winter. The existance of the aerobic zone indicated the possibility that the anaerobic pond was not loaded to maximum capacity, in terms of odors and nuisance conditions. The aerobic zone persisted on the surface of the anaerobic pond at an increased loading of 1280 lbs BOD$_u$/acre/day and flow rate of 20 gpm. However, when the flow rate was increased to 35 gpm, the aerobic surface layer disappeared and odors were soon detectable. The aerobic zone was quite effective in preventing odors, during periods of intense anaerobic fermentation, when the pond received a load of 1280 lbs BOD$_u$/acre/day or less.

Organic removal efficiencies established analytically by BOD$_5$, COD and TOC were considered the primary means of evaluating the performance characteristics of the different waste stabilization pond systems. Mean monthly values of BOD$_5$, COD and TOC data gathered during the initial test period at a loading of 70 lbs BOD$_u$/acre/day to the primary facultative ponds are summarized in Table 4.

The overall organic removal efficiency of the different pond systems was appreciably the same at their design loading. BOD$_5$ removal was in excess of 90% as calculated from filtered samples. COD and TOC removals were quite similar to each other and in the 50 to 70% range, which seems to indicate that a considerable fraction of the organic material in the pond effluents was not susceptible to microbial degradation.

A closer look at the data from Pond System I shows that a major portion of the removal of organic materials was accomplished in the small anaerobic pretreatment pond. BOD$_5$ removal exceeded 50%, and COD and TOC removal was better than 30%, which accounted for approximately half of the overall removal of these parameters. It should also be noted that the total BOD$_5$ of the system increased in the maturation pond, although the filtered BOD$_5$ dropped slightly. This phenomena was also observed in the other two systems, however, the COD and TOC values remain essentially unchanged in the maturation ponds.

Second Loading at 140 lbs BOD$_u$/acre/day

Since a deterioration in pond performance did not become evident during winter operating conditions, it was decided to double the loading before the winter season ended. Sewage flow to each pond system was increased to 29,000 gal/day during the latter part of February. The increased flow did not have an immediately visible effect on pond performance nor effluent quality.

At the increased loading, pH and suspended solids data showed only minor changes. Mean monthly pH and

Table 2. Mean Monthly Temperature Values (°C)

Month	Pilot-Scale Ponds		Lab-Model Ponds	
	Water*	Ambient	Water*	Ambient*
June 1969	--	27	30	35
July	36	30	31	36
August	38	29	32	36
September	--	27	29	34
October	14	20	25	28
November	15	14	19	23
December	13	12	18	18
January 1970	4**	7	13	14
February	--	12	17	18
March	--	13	19	20
April	--	20	23	24
May	--	--	24	25

*Single daily measurements averaged for each month.
**Single measurement.

Table 3. Summary of Mean Monthly pH and Suspended Solids Data - Pilot-Scale Ponds

Pond System	First Loading Period*		Second Loading Period**	
	pH	Suspended Solids (mg/l)	pH	Suspended Solids (mg/l)
Raw Sewage	7.3	264	7.8	218
Anaerobic I	7.2	88	7.4	58
Facultative I	8.9	70	8.7	90
Maturation I	8.7	55	9.0	98
Facultative II	9.0	104	8.2	101
Maturation II	9.4	83	8.5	97
Facultative III	8.8	99	8.6	139
Maturation III	8.9	92	9.0	106

*70 lbs BOD/Acre/day
**140 lbs BOD/Acre/day

Table 4. Summary of Mean Monthly Data - Pilot-Scale Ponds, Test Period // 1 (June 1969-January 1970)

Pond System	5 - Day BOD				Chemical Oxygen Demand				Total Organic Carbon			
	Total		Filtered		Total		Filtered		Total		Filtered	
	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)
Raw Sewage	259	--	112	--	467	--	163	--	130	--	61	--
Anaerobic I	104	60	56	50	197	58	103	37	66	49	42	31
Facultative I	22	91	13	38	130	72	79	52	41	68	29	52
Maturation I	35	86	10	91	131	72	70	57	48	63	28	54
Facultative II	39	85	13	88	167	64	81	50	60	54	30	51
Maturation II	52	80	10	91	168	64	70	57	57	56	30	51
Facultative III	34	87	10	91	160	66	72	56	59	54	29	52
Maturation III	36	86	10	91	158	66	75	54	60	54	29	52

suspended solids data obtained at the higher loading are also included in Table 3. The pH values, although they increased slightly, did not change significantly. The concentration of suspended solids, however, showed an increase of from 10 to 20% in the three different pond systems. An increase in suspended solids (mostly algal cells) was expected due to the increase in nutrients available for bacterial oxidation, fermentation and algal metabolism. It seems strange that the increase in suspended solids was not commensurate with the increase in load. The only possibility of an explanation is that the load on the ponds was still far from taxing the capacity of the systems.

Although the pond systems showed some differences in the removal efficiencies obtained at the higher load, the differences appear to be due primarily to the fact that the influent raw sewage, on the basis of the BOD data, appeared to be weaker during the second loading period. It seems evident that a change in technicians was responsible for the substantial drop in BOD_5 of the pond effluents since the COD and TOC values remained within the range of those obtained during the initial loading period. BOD_5 removal remained at about 80% and 90% for total and filtered samples, respectively TOC levels remained essentially unchanged and COD concentration were on the order of 10 to 30 mg/1 lower in the effluents of most of the ponds. The organic data for the second test period is summarized in Table 5.

Lab-Scale Pond Systems

A set of scaled down laboratory model ponds were operated concurrently with the pilot-scale ponds. Commercially available aquaria of 15 and 5 gallon capacity were used as facultative and maturation units, respectively. Table 6 is a summary of the more important operational features of the Lab-Scale Pond Systems. The lab units were operated under three different loading conditions: 1) test period 1 and 1a at 67% of the design capacity, 2) test period 2 at 133% design capacity, and 3) test period 3 at 267% design capacity. The above values are given with reference to the hydraulic design capacity. Due to the low organic content of the raw sewage available, it was felt that the lab-scale ponds would fail hydraulically if they were submitted to a flow rate high enough for them to receive an organic surface load equivalent to that of the pilot-scale ponds. The lab units were illuminated artifically receiving about 450 ft-c at the water surface.

The aquaria were fitted with over and under baffles, air jets to simulate wind action, and a combination of fluorescent and incandescent lights in a ratio of 2:1, respectively. Lab-Scale Pond Systems I and II are shown in Figures 2 and 3, respectively. Lab-Scale Pond System III is similar to System I except for the omission of the anaerobic pretreatment unit. The lights were on a daily cycle illuminating the ponds for a continuous 14 hour period each day. The units were batch fed and measured volume of raw sewage daily. Raw sewage was obtained daily from the grit chamber of the sewage treatment plant during the approximate period of peak daily flow, which corresponds to the period of high organic content of the raw sewage.

First Loading: 8 lbs BOD_u/acre/day

The lab-units were started up with a batch of algae laden water obtained from a waste stabilization pond located nearby. The model pond systems were then fed one liter of raw sewage daily. This volume of feed resulted in a hydraulic load of about 67% of the original design load and an organic load of about 8 lbs BOD_5/acre/day. The units were operated during a winter period (November 1968 – March 1969) and a summer period (June – September 1969).

At such a low loading rate the pond systems did not exhibit any differences in performance. Suspended solids and pH levels were within the same range of values as were recorded in the pilot-scale ponds. Data for the winter period (test period #1) is shown in Table 7. BOD_5 removal efficiencies were in excess of 90% for all three systems. Total COD removal in Systems I and III reached approximately 50%, but System II merely attained 29%.

Table 5. **Summary of Mean Monthly Data - Pillot-Scale Ponds, Test Period // 2 (February-May 1970)**

Pond System	5 - Day BOD				Chemical Oxygen Demand				Total Organic Carbon			
	Total		Filtered		Total		Filtered		Total		Filtered	
	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)
Raw Sewage	156	--	67	--	367	--	85	--	78	--	36	--
Anaerobic I	53	66	16	76	155	58	60	29	35	55	30	17
Facultative I	23	85	2	97	145	60	55	35	45	42	27	25
Maturation I	22	86	2	97	150	59	54	36	46	41	26	28
Facultative II	24	85	5	92	164	55	63	26	49	37	30	17
Maturation II	22	86	3	95	157	60	60	29	44	43	28	22
Facultative III	36	77	2	97	157	60	62	27	50	36	28	22
Maturation III	47	70	2	97	179	51	53	38	53	32	25	30

Table 6. **Physical and Operational Characteristics of the Lab-Model Waste Stabilization Ponds**

Pond System	Depth (in)	Surface Area (ft^2)	Flow Rate (l/d)	Detention Period (days)	Surface Loading (lbs BOD_u/Acre/day)
Anaerobic I	16	----	1	6	--
			2	3	--
			4	1.5	--
Facultative I	12	3.34	1	90	8
			2	45	15
			4	23	30
Maturation I	8	----	1	18	--
			2	9	--
			4	4.5	--
Facultative II	12	3.34	1	90	8
			2	45	15
			4	23	30
Maturation II	8	----	1	18	--
			2	9	--
			4	4.5	--
Facultative III	12	3.34	1	90	8
			2	45	15
			4	23	30
Maturation III	8	----	1	18	--
			2	9	--
			4	4.5	--

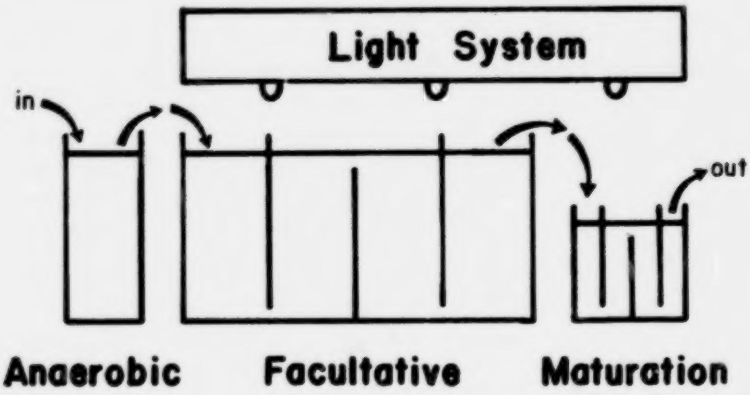
Light System
in
out
Anaerobic
Facultative
Maturation

Figure 2. Laboratory Model Ponds (System I)

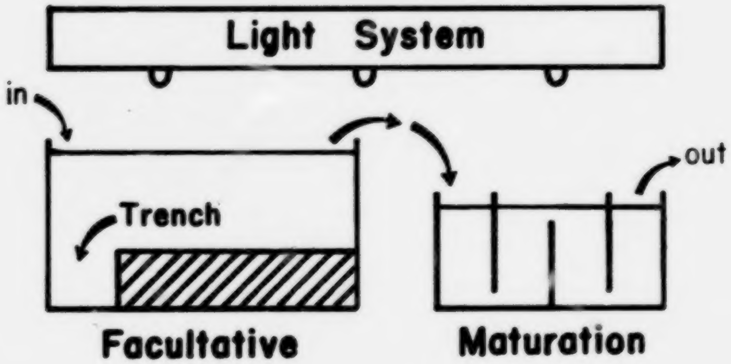
Light System
in
out
Trench
Facultative
Maturation

Figure 3. Laboratory Model Ponds (System II)

Table 7. Summary of Mean Monthly Data - Lab-Model Ponds, Test Period // 1 (November 1968-March 1969)

Pond System	pH	Suspended Solids (mg/l)	Chemical Oxygen Demand				BOD$_5$	
			Total		Filtered		Filtered	
			(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)
Raw Sewage	7.4	224	373	--	128	--	63	--
Anaerobic I	7.6	39	143	62	91	29	5	92
Facultative I	8.7	127	204	43	74	42	5	92
Maturation I	8.4	111	189	49	75	41	4	93
Facultative II	8.6	202	322	14	88	31	5	92
Maturation II	8.2	182	270	29	73	43	3	95
Facultative III	8.6	131	188	50	62	52	5	92
Maturation III	8.4	98	164	56	62	52	4	93

Table 8. Summary of Mean Monthly Data - Lab-Model Ponds, Test Period // 1a (June - September 1969)

Pond System	pH	Suspended Solids (mg/l)	Chemical Oxygen Demand				Total Organic Carbon				BOD$_5$	
			Total		Filtered		Total		Filtered		Filtered	
			(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)
Raw Sewage	7.1	285	468	--	163	--	155	--	73	--	112	--
Anaerobic I	7.5	60	160	66	104	36	104	33	54	26	---	--
Facultative I	8.7	38	90	81	68	58	44	71	32	56	---	--
Maturation I	8.9	32	89	81	70	57	37	76	32	56	---	--
Facultative II	8.9	83	120	75	70	57	54	65	35	52	---	--
Maturation II	8.7	38	92	80	65	60	40	74	29	60	---	--
Facultative III	8.6	57	74	84	57	65	45	71	31	58	12	89
Maturation III	8.8	27	76	84	70	57	42	73	32	56	9	92

The reduced efficiency of System II is explained by the higher concentration of suspended solids in the unit. System II, it will be recalled from Figure 3, lacked the over and under baffles. The absence of the baffles permitted the development of a highly stratified environment with a more uniformly mixed aerobic zone in the upper portion of the aquaria. This environment provided for a richer algal growth and, during periods of environmental stress, for a less sensitive system.

Summer operation (test #1a) was initiated in June 1969, after the units had remained idle for almost three months. The units were restarted with fresh pond water rich in algae, but the algal concentrations rapidly dropped to about 30 to 50 mg/l and remained at that level throughout the duration of the test period. Data gathered during test period #1a are presented in Table 8.

The organic removal characteristics of the lab-scale ponds were quite similar during both winter and summer operation, when considering filtered samples. Removals of 90% and about 50% were recorded for BOD$_5$ and COD, respectively, of filtered samples. Total COD removal efficiency nearly doubled from slightly more than 40% to about 80%. The low suspended solids concentrations in the effluents of the ponds during the summer period were directly responsible for the increase in efficiency.

At this stage, performance of the lab-scale units was similar to the pilot-scale ponds. BOD$_5$ of the filtered effluents from both averaged about 10 mg/l. COD and TOC of the filtered effluents averaged about 70 and 30 mg/l, respectively, for both ponds and aquaria. The only differences were in the figures for total COD and TOC where the suspended solids levels in the various units had a dominant effect.

Second Loading: 15 lbs BOD$_u$/acre/day

The loading to the lab-scale ponds was increased by a factor of two in an attempt to determine the maximum load capacity of the different systems. The increased flow rate brought the organic load to the facultative units to 15 lbs BOD$_u$/acre/day and resulted in a decreased detention period of 45 days.

During the second loading (test period #2) the aquaria regained a normal algal growth with suspended solids concentrations in a range from 100 to 200 mg/l. The pH values remained substantially unchanged, although, a higher algal level existed in the ponds. BOD$_5$ removals, as measured on filtered samples, were still in excess of 90%, which seemed to indicate that the maximum potential of the ponds had not been attained.

The soluble organic content of the ponds appeared unchanged during test period #2. COD and TOC concentrations of filtered samples averaged about 70 to 30 mg/l. However, due to the higher algal concentration, the total COD and TOC concentrations were considerably higher than in the previous test period. Mean monthly data for test period #2 is presented in Table 9.

Third Loading: 30 lbs BOD$_u$/acre/day

The hydraulic flow rate to the lab-scale pond systems was increased once more by a factor of two, in an effort to see if the maximum load capacity could be reached. The resulting load to the lab-scale ponds was 30 lbs BOD$_u$/acre/day at a detention period of 23 days. This third test period was conducted during the latter half of the winter (February to May 1970).

Shortly after the third test period was initiated, some of the lab-scale ponds developed a slightly pink coloration of the liquid. Within one month, all three pond systems had turned a dark pink color characteristic of purple sulfur bacteria. Algae were still present, but in such low numbers that they were completely blanked out by the pink coloration of the purple sulfur bacteria.

As a result of the lower algal concentrations in the lab-scale ponds, the pH level dropped one unit from 9 to 8. The high concentration of purple sulfur bacteria gave rise to an increase in suspended solids in the ponds to about 250 mg/l. Although the pond liquid was extremely murky and appeared to be of poor quality, BOD$_5$ measurements on

Table 9. Summary of Mean Monthly Data - Lab-Model Ponds, Test Period // 2 (October 1969-January 1970)

Pond System	pH	Suspended Solids (mg/l)	Chemical Oxygen Demand				Total Organic Carbon				BOD_5	
			Total		Filtered		Total		Filtered		Filtered	
			(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)
Raw Sewage	7.6	245	278	--	78	--	118	--	56	--	112	--
Anaerobic I	7.3	37	73	74	58	26	40	66	33	41	40	64
Facultative I	9.1	102	127	54	65	17	59	50	27	52	4	96
Maturation I	9.2	142	137	51	79	+ 1*	67	43	32	43	7	94
Facultative II	9.0	211	169	39	100	+28	70	41	40	29	9	92
Maturation II	8.7	139	184	34	81	+ 4	61	48	35	38	6	95
Facultative III	9.1	110	98	65	72	8	51	57	30	46	5	96
Maturation III	9.0	78	112	60	65	17	51	57	31	45	4	96

*+ indicates increase in parameter concentration.

Table 10. Summary of Mean Monthly Data - Lab-Model Ponds, Test Period // 3 (February - May 1970)

Pond System	pH	Suspended Solids (mg/l)	Chemical Oxygen Demand				Total Organic Carbon				BOD_5	
			Total		Filtered		Total		Filtered		Filtered	
			(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)	(mg/l)	(% Rem)
Raw Sewage	7.8	218	367	--	85	--	78	--	36	--	67	--
Anaerobic I	7.3	72	165	55	75	12	40	49	30	17	34	49
Facultative I	8.0	128	255	31	86	+ 1*	68	13	36	0	4	94
Maturation I	8.0	122	270	26	91	+ 7	71	9	41	+14	4	94
Facultative II	7.9	190	394	+ 7	121	+42	101	+30	48	+33	6	91
Maturation II	7.7	240	366	0	127	+49	105	+35	50	+39	5	92
Facultative III	7.8	280	334	9	80	6	89	+14	37	+ 3	4	94
Maturation III	7.9	254	346	6	92	+ 8	86	+10	44	+22	4	94

*+ indicates increase in parameter concentration.

filtered samples revealed a very low soluble oxygen demand. However, COD and TOC measurements on filtered samples revealed the presence of a relatively high oxygen demand and organic carbon content. All three pond systems showed a net gain in COD and TOC for filtered samples as well as some total samples. Mean monthly data collected during test period #3 are presented in Table 10.

The appearance of the purple sulfur bacteria seems to be a result of the combination of cold weather and increased organic loading. Once warm weather had persisted for more than two weeks, the purple sulfur bacteria started to disappear and by the time the test period had ended in the latter part of May, the ponds had completely recovered from the purple sulfur bacteria.

The organic removal efficiencies of filtered effluents were computed with respect to the soluble organic fraction of the influent raw sewage. Total organic removal efficiencies were calculated from total influent and effluent samples. The above procedure was followed in order to determine the net reductions of both the soluble and total components of the influent raw sewage. However, it is recognized that the nature of the material entering the pond was drastically altered during its residency within the pond environment.

The composition of the pond effluents was primarily algal cells in the suspended fraction and algal and bacterial respiration by products in the soluble fraction. The oxygen demanding characteristics of viable and dead algal cells have been investigated (Fitzgerald, 1961). And, apparently, the biological oxygen demand of the soluble material in the pond effluents is very low (less than 10 mg/l BOD_5). However, the effects that these pond effluents would have on a receiving stream remain to be assessed. Even though the BOD_5 of the effluents is low, they still contain a large amount of carbonaceous material, which could, at a later time, exert an oxygen demand on the stream. In addition to the carbonaceous material, measurable amounts of nitrogen and phosphorous (on the order of 5 and 10 mg/1, respectively) remain in the effluents, which could, potentially, cause eutrophication problems in the form of severe algal blooms at some point downstream from the outfall.

CONCLUSIONS

From the experimental data obtained during this study it was possible to draw the following conclusions.

1. Pond design for a particular geographical location can be improved through the use of experimental pilot-scale and laboratory units. Organic and volumetric loadings, as well as physical arrangements, for full-scale waste stabilization ponds can best be defined through the use of pilot-scale units. While laboratory units can be used to provide survey data, these small model ponds seem to be more sensitive to environmental stresses than the larger prototype systems.

2. Higher suspended solids removals and bacterial reductions indicate that facultative waste stabilization ponds function better if used in series with an anaerobic pretreatment pond. A system such as this was described in the text as Pond System I.

3. Incorporation of an anaerobic sludge deposition zone within a facultative pond, Pond System II, did not result in improved pond performance. In fact, lower COD and TOC removals were observed in Pond System II than in Pond System III, the conventional facultative pond. The lower efficiency of Pond System II was probably a result of short-circuiting due to the location of the influent pipe in the center of the pond.

4. A conventional facultative pond should be able to provide 90% BOD_5 removal for domestic sewage at a loading of 100 to 140 lbs BOD_u/acre/day and a temperature range of 12°C to 24°C. The actual loading will depend on SO_4 ion content, solids load, physical shape, and other local factors.

5. Series pond construction seems to be more important than the specific location of the anaerobic sludge deposition zone in so far as the removal of organic material is concerned. A reduction of 50 to 76% of organic material occurred in the anaerobic pretreatment unit of Pond System I. The detention period varied from three to five days and temperature ranged from 12°C to 24°C.

6. Laboratory-scale ponds are more sensitive to environmental stresses than field units. This fact is seen in the decrease of COD and TOC removal efficiencies by the lab-scale ponds at the higher loading rates and lower temperatures.

7. The lab-scale ponds without baffles gave rise to a higher concentration of algae in the effluent. A more uniform distribution of algal nutrients results in the unbaffled aquaria, due to the fact that the upper aerobic zone is fairly well mixed. Hence, the fermentation products from the anaerobic sludge layer, primarily carbon dioxide, are uniformly mixed throughout the aerobic zone.

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THEORY OF AERATED LAGOONS

*Henry B[illegible]es, Jr.***

INTRODUCTION

Aerated lagoons were a predictable outgrowth of the oxidation pond concept of natural purification. When the organic loading rates on oxidation ponds exceeded the limit that nature had intended, machinery was used to artifically induce oxygen at higher rates.

The term *aerated lagoon* is a general description, applying to a variety of processes. The variety results because in the conversion of oxidation ponds to aerated lagoons, different engineers applied varying degrees of artificial mixing and oxidation to solve the problem of overloaded oxidation ponds.

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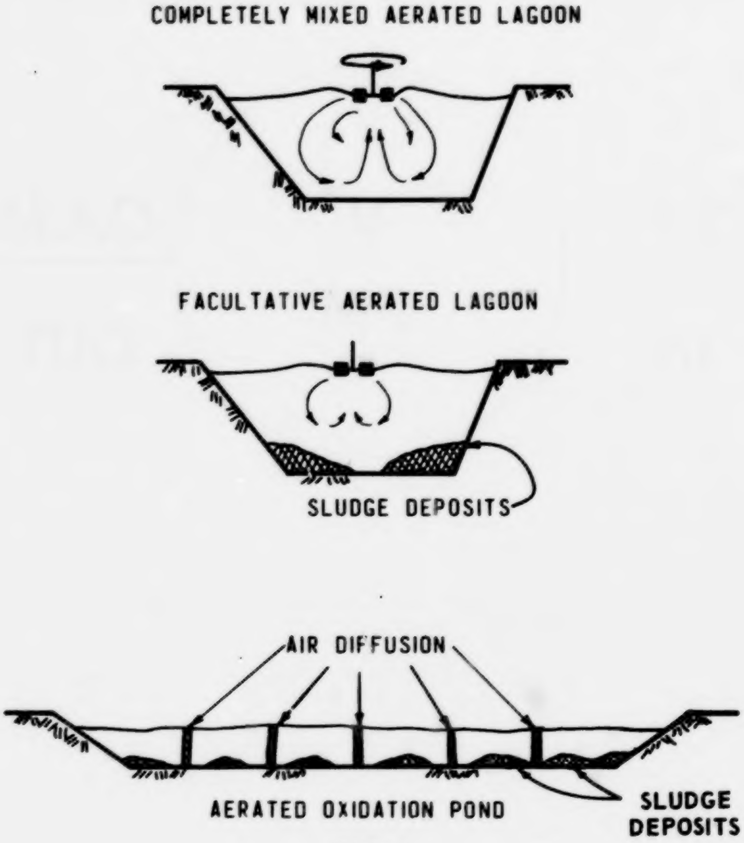


Figure 1

Shown on Figure 1 are three distinctly different types of aerated lagoons:

Completely Mixed Aerated Lagoon

The completely mixed aerated lagoon is an activated sludge aeration basin without sludge return where all solids are maintained in suspension. All organic stabilization is dependent upon the suspended bacterial solids. Oxygen is machine derived and algae are not apparent.

Facultative Aerated Lagoon

The facultative aerated lagoon has a degree of mixing which permits heavier solids to settle in the lagoon while biologically treating suspended and soluble organics in the liquid portion. The oxygen transfer is primarily associated with the machine. Algae may be present in small numbers but play only a small or insignificant role in the oxygen supply.

Aerated Oxidation Pond

The aerated oxidation pond is essentially a high rate oxidation pond. Mixing and aeration equipment is added to an existing overloaded oxidation pond, distributing the high oxygen concentration near the surface throughout the pond contents. Oxygen is primarily transferred by algae except during cold months.

DESIGN DEFINITIONS

The design of aerated lagoons centers upon the creation of a favorable environment for the micro-organisms. To create this favorable environment, an understanding of the biochemical reactions involved is necessary. The mathematical definition of the biological reactions has attracted many investigators. Unfortunately, there are nearly as many approaches as there are investigators.

The mathematics of the complete mixing aerated lagoon have been the primary area of activity because of the relative ease of process definition. The major sets of mathematical analyses were developed by Eckenfelder[1], McKinney[2], and Monod[3]. This presentation will not attempt to resolve the differences in these equations but merely will discuss the major differences between each.

The typical process schematic for the completely mixed aerated lagoon is shown on Figure 2. The influent organic concentration (Fi) and flow enter the aerated lagoon and are "immediately" dispersed throughout the basin; bacterial solids are synthesized from the available food and maintain a depressed organic concentration level in the aeration basin. The amount of oxygen required is

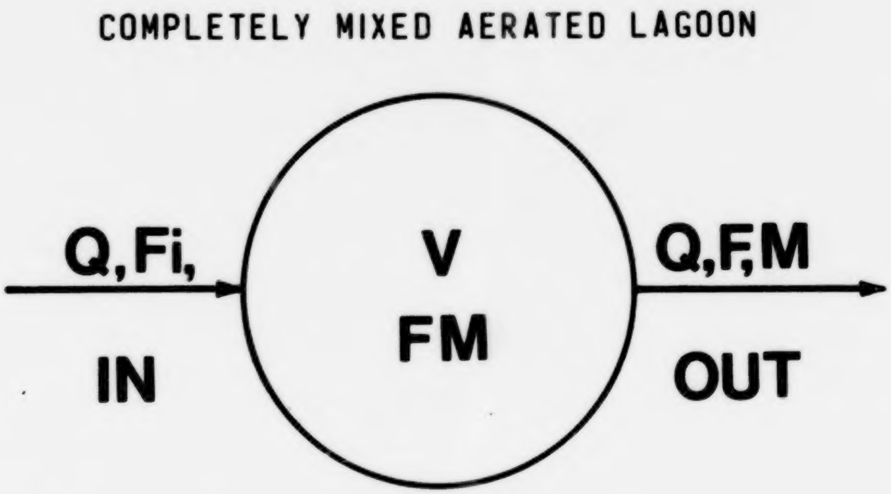


Figure 2

Table 1 – Comparison of Mathematical Equations For Aerated Lagoons

	Eckenfelder	*McKinney*	*Monod*
Soluble effluent BOD	$\frac{Fi}{kr\ Mv\ t + 1}$	$\frac{Fi}{kr'\ t + 1}$	$\frac{\Delta F\ Ks}{k\ Mv\ t - \Delta F}$
Volatile solids produced	$\frac{ks\ (Fi - F)}{1 + ke\ t}$	$\frac{ks'\ F\ t}{1 + ke'\ t}$	$\frac{ks\ (Fi - F)}{1 + ke\ t}$

Where

kr, kr'	– removal rate constant
k	– maximum substrate utilization rate / cell mass
Ks	– half velocity constant (substrate concentration when dF/dt = ½ K)
ks, ks'	– synthesis constant
ke, ke'	– endogenous constant
Fi	– influent BOD
F	– effluent BOD
ΔF	– Fi - F
t	– detention time
Mv	– mixed liquor suspended solids

dependent upon the amount of food used and the detention time.

The effluent from the basin is identical to the liquid in the aeration basin and contains the synthesized bacterial solids, the remaining soluble effluent BOD, and any inert solids carried in the effluent. The effluent BOD is the remaining soluble BOD and the BOD due to bacterial solids.

Table 1 shows the basic equations for calculating the soluble effluent organic substrate concentration (BOD) and the volatile solids production for aerated lagoons. The equations presented and the symbols used in the referenced work have been rearranged to develop similarity for comparative purposed. Modified Monod equations were presented in a recent publication[4] and the forms used in that publication were used to assemble the equation forms shown in Table 1.

For calculating the soluble effluent organic concentration, the Eckenfelder and Monod equations take the same general form where soluble effluent BOD is a function of bacterial solids mass, detention time and influent organic concentration. Eckenfelder's calculation is a function of the raw organics while Monod's equation is a function of the usage rate of organics. The rate of organic use is dependent upon the food remaining. Comparing McKinney's equation with that of Eckenfelder, the difference lies in the absence of any dependence on the

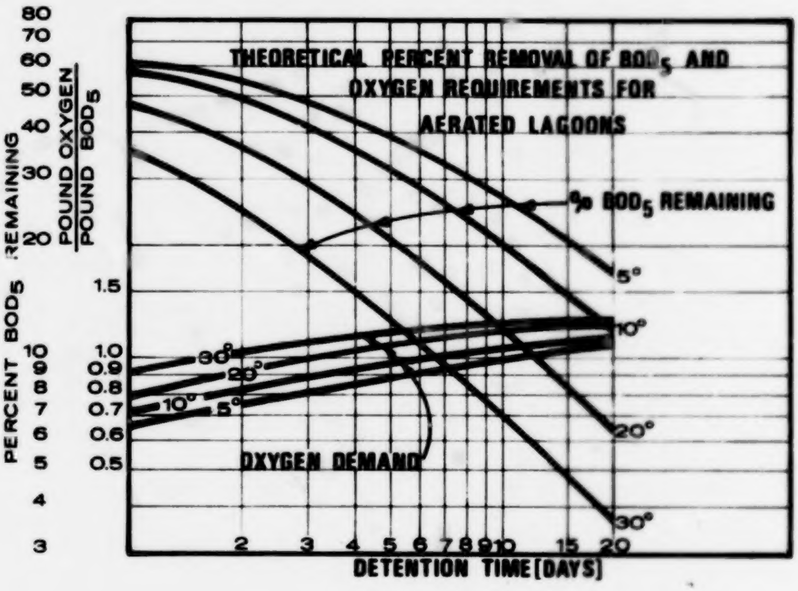


Figure 3

aeration solids in McKinney's equation.

For calculating the aeration basin volatile solids production, Eckenfelder and Monod have used the same equation form. McKinney has developed an equation on the basis that the active microbial mass is dependent upon the food remaining (F) rather than on the yield per amount of food utilized (Fi-F). Essentially the Eckenfelder and Monod equations use change in cell production per unit of time as a function of the rate of food used per unit of time. The Monod equation at low substrate concentration simplifies to state that the rate of food used per unit of time is a function of the substrate concentration. McKinney has stated this directly in the derivation of his equation; therefore, these equations have similar foundations.

An additional difference unique to McKinney's equations, not shown in Table 1, is that the formula for calculating the volatile solids production only applies to the active mass portion of the volatile solids produced, whereas the Monod and Eckenfelder equations are used to calculate total volatile solids production. McKinney has related the nonviable volatile solids formed in endogenous respiration to the active portion and the detention time.

$$Me = 0.2\ Ke\ Ma\ t$$

Disagreement prevails among engineers as to which of the above equation sets is absolutely correct; however, when real and good data are applied to these equation sets, adequate correlations usually are found because each has been developed from real data or has been tested extensively against real data. Confusion develops from the fact that they are different; seemingly every time they are presented, different symbols are used and the differences in the equation forms cause different constant values to be used in the equations.

Because the author's firm generally uses McKinney's equations for data evaluation and calculations, this equation set has been further developed for completely mixed aerated lagoon design relationships. Figure 3 shows the relationship between the per cent reduction of influent BOD_5 and oxygen utilization against detention time for varying temperatures. The oxygen demand has been calculated based on an oxygen equivalence balance[5] and assuming domestic wastewater characteristics:

$$\frac{dO}{dt} = [1.5\ \Delta F - 1.42\ (Ma + Me)]\ /t$$

Where:

$\frac{dO}{dt}$ = oxygen demand expressed as lb/pound of applied BOD

ΔF – substrate used

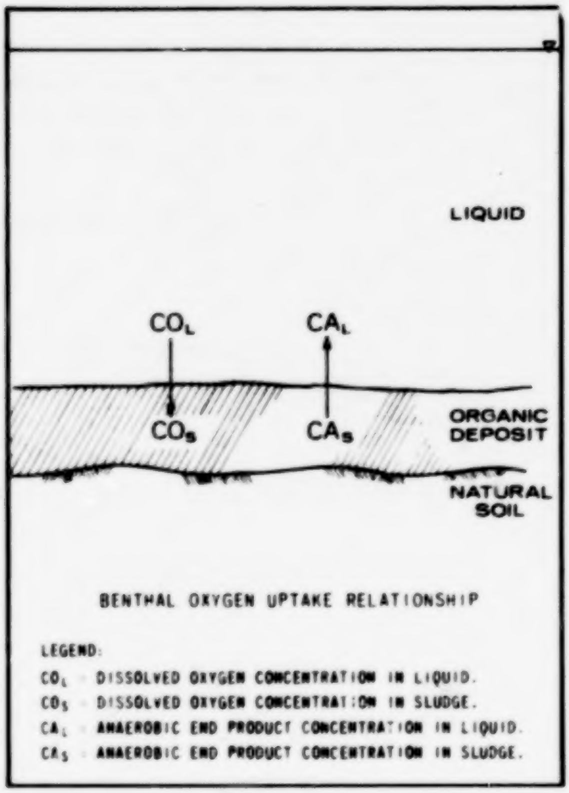
Ma, Me – active and endogenous solids production

t – detention time

The work done on completely mixed aerated lagoons generally uses a day or more detention time in the aeration cell.[6,7,8] There is a minimum detention time in aerated lagoons below which process failure will result. Process failure occurs when the hydraulic washout rate of the bacteria exceeds the synthesis rate. Lawrence and McCarty[4] report a minimum detention time for glucose substrate as being 0.12 to 0.85 days. This compares with studies by McKinney and Ooten[5] where the limiting detention time was about 0.13 days for glucose substrate. The minunum detention time is temperature dependent and will increase with lower temperatures.

Facultative Aerated Lagoons

In an aerated lagoon where a portion of the solids is removed by sedimentation, the mathematical definition of



BENTHAL OXYGEN UPTAKE RELATIONSHIP

LEGEND:

CO_L - DISSOLVED OXYGEN CONCENTRATION IN LIQUID.

CO_S - DISSOLVED OXYGEN CONCENTRATION IN SLUDGE.

CA_L - ANAEROBIC END PRODUCT CONCENTRATION IN LIQUID.

CA_S - ANAEROBIC END PRODUCT CONCENTRATION IN SLUDGE.

Figure 4

the results from the lagoon is elusive. The organic loading on the aerated lagoon will vary depending on the amount of settleable solids in the raw waste.

A model and prototype study of a facultative aerated lagoon were described for a subdivision installation in suburban Kansas City, Missouri.[8]

Mixing was induced with a turbine type mixer at 1/11 hp/1,000 cu ft. The study concluded that incomplete mixing was developed and bottom deposits resulted.

The solids on the bottom are decomposed by benthal oxidation (see Figure 4), meaning, the surface of the deposit is oxidized aerobically and the level beyond which oxygen does not penetrate into the deposit is oxidized anaerobically. Liquefied anaerobic end products diffuse from the sludge deposit and are oxidized anaerobically in the liquid phase. The benthal oxygen demand rate is dependent upon the depth of sludge deposit and upon the fluid velocity over the deposit.

Studies of benthal oxygen demand rates on the overlying fluid have been reported and formulated by others.[9, 10]

The findings of the study of the Red Bridge facultative aerated lagoon were that the oxygen demand in the fluid portion was lower than a complete mixing lagoon. This would be expected by the removal of organics by sedimentation. The effluent BOD was lower than would be predicted for a complete mixing aerated lagoon, again a result expected if organics were removed by another means. The same conclusion resulted from a study at the Chemawa Indian School waste treatment system.[11]

One caution from the Red Bridge study concerned odors. An odor level was apparent during the early spring in the vicinity of the lagoon.

Eckenfelder[1] also studied facultative aerated lagoons and reported that the soluble effluent BOD can be approximated by an equation form very similar to McKinney's shown in Table 1, however, using the kv value developed from the basic Eckenfelder equation. Eckenfelder also states that the solids level in the aerated lagoon is not predictable due to sedimentation.

Facultative lagoons require less power for mixing and aeration than complete mixing aerated lagoons and more effective use of the detention time is made since removal by sedimentation permits lower levels of aeration soluble substrate. The disadvantage of the facultative aerated lagoon is that it does not lend itself to mathematical prediction as well as the complete mixing variety and therefore must be evaluated more empirically. Also the problem of odors associated with the facultative lagoon has not been fully solved.

Aerated Oxidation Pond

The aerated oxidation pond oxygen supply is primarily algae dependent in the warmer months. In many cases aeration has been added to an existing overloaded lagoon to supplant the natural oxygen resources. However the beneficial effect of the aeration device is more related to the mixing of the pond contents which better distributes the oxygen throughout the liquid phase. Algae in non-mixed oxidation ponds typically supply oxygen near the surface of the liquid. The oxygen diffuses downward, while the anaerobic end products from the sludge layer diffuse upward and an oxygen gradient from supersaturation at the surface to zero at some depth in the lagoon occurs. The addition of mixing reduces this gradient and more effective use of the oxygen supplied by the algae is made.

There have been several aerated oxidation ponds mentioned in the literature, using a variety of mixing devices. One of the most prevalent is gas diffusion through a plastic hose distribution system.*

McKinney[12] reported on two such systems in the Kansas City area. One lagoon was 25 acres by 5 feet deep and had an estimated organic loading of 100 pounds per acre. The air supply furnished was 1.3 cubic feet per square foot of area per day or 570 cubic feet per pound of applied BOD. The median influent BOD concentration was 125 mg/l and the median effluent BOD was 25 mg/l or 79 per cent removal. Some odors were reported to be apparent most of the time.

The catalog design criteria for the plastic tube aeration of oxidation ponds indicate the following parameters:[13]

Depth	Air Applied Per Pound of BOD	Air Applied Per 1,000 Cubic Feet of Pond Capacity	Detention	BOD Per Acre Per Day
feet	cu ft	cfm	days	lb
10	720	0.25 to 0.50	30*	200*
5	900	0.25 to 0.33	30*	100*

Another form of aerated oxidation pond was presented by Jenks.[14] An "inka"** system was installed in a heavily loaded lagoon to aerate and circulate the lagoon contents. The pond was 1/3 acre and had an estimated loading of 340 pounds of BOD_5 per day. The inka system airlifted a flow volume of about 4,000 gpm. The amount of oxygen transfered by the inka system was 72 pounds per day and the oxygen transfer by algae made up the balance.

A third type of aerated oxidation pond is located in Lenexa, Kansas using three 5 horsepower high speed floating aerators. The lagoon is loaded at approximately 380 pounds of BOD_5 per acre per day (0.75 acres). Measurements of dissolved oxygen throughout the lagoon showed no discernible gradient. The aerated lagoon has exhibited an average BOD_5 removal rate of 68 per cent.

The aerated oxidation ponds are designed on an empirical basis. Solids will settle out in the basins and exert a benthal oxygen demand. Depending on the oxygen concentration in the basin, the aeration devices may or may not transfer significant oxygen quantities; typically, the algae in the system will transfer the great portion of the oxygen and the aeration device will serve primarily as a means to circulate the pond contents. The mixing of the lagoon contents is as important a parameter as is the oxygenation.

There appears to be a tendency in some areas to look unfavorably on oxidation ponds because of the algae produced which in turn exit in the effluent and are measured as BOD and suspended solids. The required removal of algae to an innocuous level may be in the offing.

Mixing

The degree of mixing is the basic difference between the various aaerated lagoon styles. To a certain extent, detention time is an affecting parameter.

The problem of quantifying the "degree of mixing" or the "goodness of mixing" to obtain the desired end result is complex. The complexities derive from the many varieties of mixer designs, diffusion equipment, differences in geometry, and the apparent lack of parameters to compare one mixing concept against another.

The amount of mixing derived from a unit of power is different for different types of mixing devices. For instance, the amount of circulating pumpage directly created by a turbine mixer and a propeller mixer are shown as related to power in Figure 5.[15]

Comparing slow moving, large mixers to high speed, small mixers, Kalinske[16] developed the following argument:

Mixing is a turbulent diffusion phenomenon.

Turbulent diffusion can be characterized as proportional to the velocity of the stream and its relative size.

The velocity at a remote point from the turbulent source can be increased more efficiently by increasing the flow at the source or less efficiently by increasing the velocity at the source.

Large, slow moving impellers have a large source flow per horsepower than small, fast moving impellers.

Therefore, large, slow moving impellers are more efficient mixing devices than small fast moving impellers.

Kalinske carried the argument further showing that large slow moving impellers are more efficient mixing devices than is gas mixing.

The use of installed machine horsepower per volume is a mixing parameter has tempted several investigators. Power dissipation as defined by the velocity gradient "G" was investigated and applied to mixing in aeration basins by Fair et al.[17] and Richard and Gaudy.[18] The mistake commonly made in using the velocity gradient is in calculating the power dissipation on the basis of the horsepower required for the machine and not the fluid power dissipation which is the process requirement. Because different mixing devices consume different amounts of power to effect a defined mixing result, the comparative use of machine horsepower per volume is not valid. One of the major results of Fair and Richard and Gaudy's studies were that mixing plays an important role in determining the biological reaction rate.

Mixing has not reached the science stage and still remains an art. The emphasis in mixing should be placed more on the end result and less on the means. With complex mixing problems and where there is a desire to compare mixing devices, it is recommended that hydraulic models be used to study the effects before the prototype is constructed.

When mixing is a consideration, as it would be with the facultative or completely mixing lagoon, consideration should be given to large impeller mixers operating at slow speeds.

Aeration

The design of the mixing equipment must also be compatible for the required oxygen transfer. The problems of defining aeration capacity and evaluating aeration

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*Hinde Co., Air Aqua

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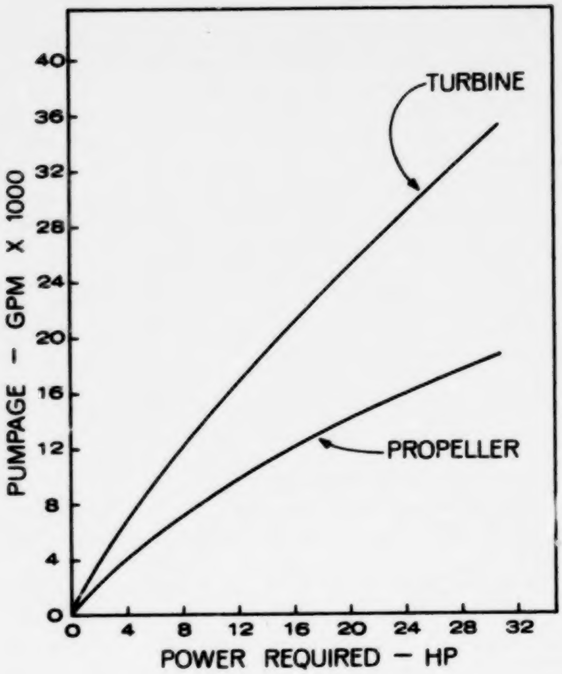

Figure 5

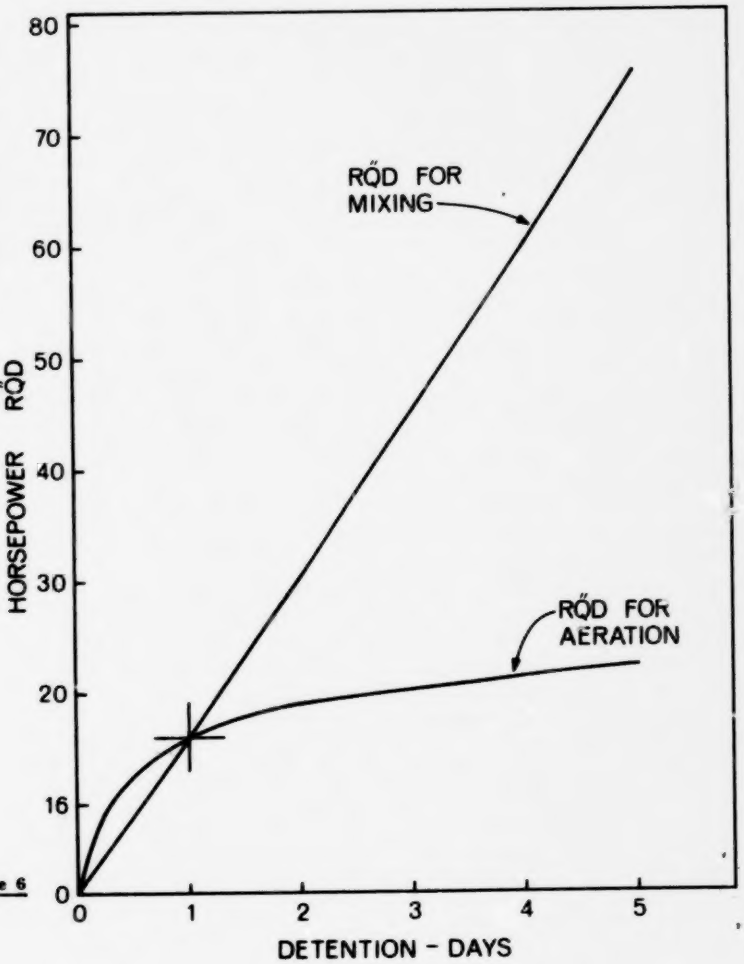

Figure 6

devices have been discussed.[19]

Given the following parameters, the information on Figure 6 was developed:

Q	0.6 mgd
BOD	1,000 lbs (200 mg/1)
Temperature	20 C
Oxygen capabilities of machine*	2 lbs/brake hp/hr
Machine pumping capacity	2,000 gpm/brake hp
Pumping requirements, turnover time	20 minutes

From the relationship shown on Figure 6, the aeration capability of the machine beyond one day detention is a secondary concern to the degree of mixing required to have complete mixing.

*In aeration basin liquid

SUMMARY

The term *aerated lagoons* has been applied to a variety of processes which in this presentation have been further defined as:

Completely mixed aerated lagoon
Facultative aerated lagoon
Aerated oxidation pond

The completely mixed aerated lagoon process has been defined mathematically by several investigators. The equations used by each have notable differences.

The completely mixed aerated lagoon mixing considerations are as critical if not more critical than the aeration considerations. The types of mixing equipment available develop different process mixing levels for the same power. The large slow moving impellers are more efficient mixing devices than either the fast moving, small impellers or gas diffusion.

The facultative aerated lagoon removes raw waste organics by aerobic stabilization and sedimentation, and the subsequent stabilization of bottom sludges by benthal decomposition. The characteristics of the facultative aerated lagoon are not easily defined and normally are designed empirically. Less power is required than for a completely mixed aerated lagoon and higher treatment efficiencies are attainable for wastes having a large portion of organic settleable solids. The odor potential has not been well established; however, it has been a problem in some installations.

The aerated oxidation pond behaves like a high rate oxidation pond. The oxygen supply is primarily from algae during the warm months. Loadings as high as 1,000 pounds per acre have been reported.

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AERATED LAGOONS IN ONTARIO - DESIGN AND PERFORMANCE CONSIDERATIONS

B. I. Boyko and J. W. G. Rupke[1]

INTRODUCTION

The use of aerated lagoons in the Province of Ontario as a wastewater treatment process is relatively new with virtually all aerated lagoons being constructed within the last decade. Most of these installations are located in Southern Ontario, an area of temperate climate with January mean daily minimum temperatures ranging from 2°F to 20°F, and July mean daily maximum temperatures of 76°F to 82°F. The majority of the aerated lagoons have been designed as complete treatment facilities with effluents being discharged to a watercourse; however, these effluents do not compare in quality with those normally associated with secondary treatment. In a few instances, aerated lagoons are used for pretreatment of high strength wastewaters before discharge to municipal sanitary sewers.

This presentation will discuss previous design criteria, various types of aerated lagoon facilities, and aeration devices currently in use. In addition, performance and operating problems of these installations will be reviewed and recommended design criteria outlined.

INSTALLATIONS AND PREVIOUS DESIGN BASIS

In describing the physical and process design data associated with these facilities, two categories of aerated lagoons will be considered.

The first general classification is the facultative aerated lagoon, i.e. allowing solids deposition while maintaining a uniform dissolved oxygen concentration throughout the liquid phase. Such lagoons have generally been designed to provide anywhere from 8 to 20 days retention. In some cases, the design has been based on a volumetric BOD loading (from 5 to 10 lb BOD/1000 cu ft/day) followed by a subsequent stabilization zone within the same basin. These loadings have resulted in retention periods of 1 to 6 days in the aeration zone. The other design consideration, oxygen supply, was usually satisfied by providing 1.2-1.4 lb 0_2/lb BOD applied.

Table I outlines the physical data associated with these facultative aerated lagoon systems. Table I also includes a column entitled Mixing Turnover Time in Minutes. This parameter is a measure of the mixing intensity within the basin. Its numerical value is calculated by dividing the volume of the basin by the pumping capacity of the aerator. When the pumping capacity calculation is based on only the flow which passes through the aeration device (not including flow associated with induced currents) a turnover time of approximately seven minutes will ensure complete mixing of biological solids.

The other type of aerated lagoon occasionally encountered is the mixed aeration lagoon, sometimes referred to as an aerobic lagoon. Here, a high level of turbulence is provided to maintain all solids in suspension, and thus no organic removal takes place through solids deposition. The design of these systems has essentially been based on retention time (up to four days) and satisfying oxygen requirements, again 1.2-1.4 lb 0_2/lb BOD applied.

The physical data associated with mixed aerated lagoon installations are also shown in Table I.

PERFORMANCE DATA ON AERATED LAGOONS

The loading conditions of the aerated lagoon systems previously described are presented in Table II, with the data being obtained during in-depth studies of each facility carried out over the past several years. These field investigations usually were of six months to one year duration. In addition to determining raw waste and effluent characteristics, dissolved oxygen profiles and solids deposition were also measured.

In Table III, the per cent BOD and suspended solids removals and associated effluent qualities for the treatment systems are summarized.

DISCUSSION

Effluent Quality

The results in Table II indicate that aerated lagoons do not provide a high quality effluent comparable to secondary treatment. While the degree of treatment is frequently quite high, the effluent is generally similar to that from waste stabilization ponds. With domestic wastes, an effluent quality of 30 to 50 mg/1 BOD and SS can be expected from a facultative aerated lagoon. However, aerated lagoon systems feature several advantages over conventional ponds. Reduced capital costs are an obvious benefit, but perhaps the major advantage is the consistent performance obtained. In our studies to date, we have not observed any seasonal fluctuation in effluent quality such as may be associated with temperature or algal effects. This relatively uniform performance could likely be attributed to conservative design parameters.

BOD Removal

A) Facultative Aerated Lagoons

The mechanism of organic removal through aerated lagoon systems warrants consideration. In a facultative system, the three significant removal mechanisms are sedimentation, biological and chemical oxidation. Bottom sludge surveys have indicated that extensive solids deposition does take place. The biological or biochemical removal appears to take place through a bacterial rather than an algal system.

Dissolved oxygen traces have indicated that the DO concentration follows the applied BOD load; as the hourly BOD applied increases, the DO decreases. In an algal system, the DO generally responds to light intensity rather than the applied organic load.

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Table I – Aerated Lagoon Physical Data

Type of Waste	Flow mgd	Detention Time Days	Oxygen Supply Design lb/day	Oxygen Supply Type	Mixing** Turnover Time min
i) Facultative Aerated Lagoons					
A) Domestic (Seasonal Winery)	0.15	8	640	Mechanical	0.05*
B) Chicken Processing	0.18	18.2	1300	Diffused Air	117
C) Potato Processing	0.75	14	2650	Diffused Air	112
D) Domestic	0.50	7.8	1430	Diffused Air	17.2
E) Instant Coffee Processing	0.278	10.6	1460	Diffused Air	18.4
F) Fruit Canning	0.125	38.6	2800	Diffused Air	28.9
ii) Mixed Aerated Lagoons					
G) Domestic	0.14	8.8	1170	Mechanical	0.10*
H) Mixed Industrial	0.18	1.2	900	Mechanical	0.57*
I) Mixed Industrial & Reformatory	0.15	3.2	640	Mechanical	0.14*

* HP/1000 ft^3

$$\text{**Turnover time in min} = \frac{\text{Aeration tank volume in gal}}{\text{Aerator pumping capacity in gal/min}}$$

Table II – Loading Conditions

	Raw Sewage			Organic Load		Hydraulic Load	
	Flow	BOD mg/l	Susp. Solids mg/l	lb BOD/ 1000 ft^3/ day	% Design	Detention Time Days	% Design
i) Facultative Aerated Lagoons							
A)	0.15	154	110	1.17	54	8	-
B)	0.18	600	368	1.75	123	18.2	109
C)	0.75	1200	400	5.3	408	14	125
D)	0.50	145	120	5.6*	61	1.6*	83
E)	0.278	540	848	13.0*	124	2.6*	93
F)	0.125	1900	236	5.1*	102	25.8*	33
ii) Mixed Aerated Lagoons							
G)	0.14	590	153	3.6	120	8.8	48
H)	0.18	352	214	18.8	85	1.2	310
I)	0.150	166	125	3.56	-	3.2	-

*Based on aeration section volume only

Table III – Effluent Quality

	Effluent			% Removal	
	BOD mg/l	Filtered BOD mg/l	Susp. Solids mg/l	BOD	Susp. Solids
i) Facultative Aerated Lagoons					
A)	25	20	40	84	64
B)	85	40	56	66	85
C)	1100	800	249	8	38
D)	30	20	50	80	58
E)	266	210	89	51	74
F)	500	400	25	74	89
ii) Mixed Aerated Lagoons					
G)	170	30	160	71	-4.6
H)	200	75	200	43	0
I)	124	28	220	25	-76

With such varied organic removal mechanisms, perhaps interrelated, the design use of an overall removal rate constant is problematical. Such rate constants are generally associated with the soluble BOD and thus ignore BOD removal by sedimentation. However, rate constants may be obtained from laboratory or pilot data and then used in an empirical plant design. In the design of aerated lagoons, the use of a volumetric organic loading factor likely has greater applicability than generalized rate constants. Field observations have indicated that at an organic loading factor of approximately 2 lb BOD/1000 cu ft/day, an acceptable effluent quality may be expected.

B) Mixed aerated Lagoons

Because of the high levels of turbulence associated with these systems, no BOD removal by sedimentation is encountered. Thus the remaining mechanisms for organic removal are chemical and biological oxidation, usually accompanied by synthesis of bacterial mass. With the high turbulence levels in these systems, the sludge age is essentially the hydraulic retention time. In the mixed system G) in Table II with retention times of 8.8 days, a flocculant activated sludge type of solids was produced. In a settling test, these solids settled readily leaving a clear, high quality supernatant. However, in systems H) and I) when the detention time is only one to three days, the lagoon effluent was turbid with no well-formed flocculant material present. The dispersed nature of the sludge produced is likely a result of the low (1-2 day) sludge age associated with these systems.

For aerated lagoon considerations, there does not appear to be any merit in providing a completely mixed system unless a clarifier is provided to increase the effective sludge age thus providing a sludge with flocculant characteristics. The system then becomes a low rate activated sludge process. Without the clarifier, the total effluent BOD will be considerably greater than that from a facultative aerated lagoon, with the major portion of the effluent BOD due to the suspended solids. This is illustrated by the BOD and filtered BOD data in Table III. In cases where a stabilization pond follows the mixed aerated lagoon, the discharge of high solids to the second pond may create a rather severe odour problem. Again, the applicability of such a mixed aerated lagoon system appears questionable, unless future conversion to the activated sludge process is anticipated.

Aeration Devices

The surface aerators commonly in use are either pier mounted or floating units having the same action as those found in activated sludge systems.

For subsurface aeration, both air gun and perforated tubing systems are in use. Considerably less maintenance has been associated with the air gun system than the perforated tubing.

Lower capital and operating costs are normally associated with surface aeration devices.

PROBLEM AREAS

With facultative lagoons, a minor problem that has been observed is floating sludge. During the spring warm-up, masses of anaerobic sludge periodically rise to the surface not only yielding a poor effluent but also causing a considerable odour problem. If surface agitation is great enough, this fragile sludge may be quickly dispersed with minimal problem.

In all aerated lagoon systems, grit, rag and debris removal should be provided to minimize long-term operating difficulties.

Reliable performance of aeration devices, either surface or subsurface, has been a significant problem in the operation of these installations.

No mechanical problems have been associated with winter operation of lagoons using subsurface aeration devices. With proper design, icing has not been a problem with surface aerators.

DESIGN CONSIDERATIONS

Perhaps the major process limitation with the use of aerated lagoons is the fundamental problem of getting a high strength waste down to a secondary or acceptable effluent quality in one single stage treatment unit. As this usually involves removal efficiencies in excess of 95%, it appears unrealistic to expect such performance from single stage units. In the past, there has been a reluctance to design multi-stage aerated lagoon systems for the treatment of high strength industrial wastes. This reluctance has led to single stage systems such as C) and F) being misapplied. Where BOD removals in excess of 90% are required, multi-stage treatment facilities should be considered.

For design of facultative aerated lagoons, a volumetric organic loading of 2 lb BOD/1000 cu ft/day should provide an acceptable effluent quality. If an attempt is made to design a facultative aerated lagoon at higher volumetric organic loadings care should be taken to avoid the high mixing conditions that may result from satisfying the associated oxygen requirements. For aerated lagoon considerations only, facultative systems are to be preferred over mixed systems.

In cases where industrial wastes or mixtures of domestic sewage and industrial wastes are to be treated, it is essential that some preliminary biodegradability studies be carried out. These tests should be of a continuous feed nature with parallel investigations being carried out to determine the nutrient requirements of the system. Care should also be taken to ensure a representative sample of the expected feed is used.

ALASKA SEWAGE LAGOONS

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There are many isolated communities and governmental installations in Alaska with populations of less than 1000 people. Most native villages are not sewered. A few of the government outposts are served by extended aeration package plants, others have septic tanks, and the rest have no treatment. Some of the septic tanks dump directly into the ground.

An aerated lagoon with its ability to handle overloads and its simplicity of operation is attractive for remote installations. Faculative lagoons with their extreme simplicity are also attractive for remote site application, expecially areas where power is unavailable or extremely expensive ($0.25 to $1.00 per kilowatt-hour).

In 1957 a small facultative lagoon was placed in operation north of the Arctic Circle at Fort Yukon, Alaska. The system is a leaky lagoon (no definite effluent line exists) that serves a school.

Late in 1967 the Alaska Air Command and the Alaska Water Laboratory entered into an agreement to construct and operate a research field facility that included an aerated lagoon pilot plant at Eielson Air Force Base, 22 miles southeast of Fairbanks in Interior Alaska. Most of the information in this report has been obtained from the Eielson lagoon.

When considering lagoons for Alaska and Northern Canada application, strong emphasis must be placed on cold weather impact.

Facultative Lagoons

The Northern Regions Sewage Lagoon Performance Statistics presented in Table 1 indicate a number of factors of importance: 1. winter BOD and solids removal efficiencies are not significantly improved by longer detention times in a single cell, but are significantly improved when a system has several cells operated in series; 2. BOD proved when a system has several cells operated in series; 2. BOD removal efficiencies are not affected by increases in lagoon depth; 3. bottom sludges will accumulate and their rate of accumulation will, in general, increase as one travels north, and the rates vary from 8.8 to 14.0 cu ft per 1000 people per day; and 4. although winter efficiencies for the removal of total coliforms are not well defined, winter removals of 97.7 to 99.9 percent were reported at Inuvik and Yellowknife. Pathogenic organisms have been isolated from the sewage deposits of early expeditions and camps in the Arctic and Antarctic many years after man's activities at the sites had ceased (2).

The actual lagoon size is directly proportional to the required system detention and the system cost is also directly proportional to the length of detention; therefore, loading rates become a highly important factor. Svore (29) found that loading rates are the key to odor control and states that "10 lbs applied BOD per acre per day usually eliminates Spring breakup odor problems; 20 lbs applied BOD per acre per day creates odor during Spring breakup of 1 to 2 weeks duration; 25-30 lbs applied BOD per acre per day creates odors of 3 to 4 weeks duration; and 50-60 lbs applied BOD per acre per day creates odors that persist for as long as 3 months". On the other hand, Brisbin (6), Dawson (7,8,9) and Higo (13) have found that odor control can be attained at much higher organic loading rates. At Moose Jaw, Saskatchewan, odor problems became so serious with a loading of 1080 lbs applied BOD per acre per day and 7 day detention that the lagoons had to be bypassed. The Sutherland, Saskatchewan lagoons operating at 1300 to 2600 lbs applied BOD per acre per day and detentions of 2.75 and 1.45 days seldom create odors detectable at distances greater than 1000 feet with detectable odors generally limited to distances less than 100 to 300 feet. Dawson (7,9) reports definite improvements at Yellowknife, Northwest Territories, when a short detention anaerobic primary cell (0.6 day detention and 8 ft depth) was added. Higo (13), after investigating a number of lagoons in Alberta, Canada, concludes that the potential for odor production can be minimized when designing short detention time systems by provision of central submerged influents, depths of 10-20 feet, detention of 2 to 4 days (not over 4 days at existing flows), and BOD loading not to exceed 9 lbs applied BOD per 1000 cu ft per day (4000 lbs applied BOD per acre per day at 10 ft depths).

Dawson (7,8,9) and Grainge (12) have found that stabilization lagoons operating under extreme winter conditions provide BOD and solids removal rates that are approximately equivalent to primary waste treatment.

Both North Dakota (31) and Minnesota (1,11) are recommending that stabilization lagoons be designed with sufficient volume to provide for retention of all winter flows; thus limiting effluent discharge to that period of the year when adequate treatment is obtained.

Aerated Lagoons

Aerated Lagoons have operated successfully in cold regions of the contiguous United States and Canada for a number of years. The first aerated lagoon in Alaska was

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Table 1. **Northern regions sewage lagoon performance statistics (Compiled From Reports and Publications Referenced in Last Line) (1)**

	Inuvik, N.W. Territories	Yellowknife, N.W. Territories before modification	Yellowknife, N.W. Territories After Modification Series Operation		Lacombe, Alberta (3140)
			Primary Cell	Secondary Cell	
Latitude, °N	68°21'	62°	62°	62°	52°
January Average Air Temperature, °F	-28.2 (Febr. -34.1)	-20.3 (Febr. -28.2)	-20.3	-20.3	Edmonton +3.7
Lagoon Type	Facultative	Facultative	Anaerobic	Facultative	Facultative
Lagoon Depth (ft.)	1.9-3.4	3.2	8.0	3.2	5.0
Detention Time (days)	73-136	134	0.6	99	202
Loading--#BOD$_5$/acre/day --#BOD$_5$/1000 ft.^{3}/day	8.9-9.6 0.082	15 0.108	5170-6170 11.9-14.2	18.8-19.4 0.136	25.7 0.118
Ice Conditions--Duration Starting Month to End Month --Depth (ft.)	October May To bottom except at inlet	October May 1-1/2	None 0	October May 1-1/2	November March 5.0
Sewage--Source --Winter Temperature, °C	Raw Domestic 19°C	Raw Domestic <6.5°C	Raw Domestic <6.5°C	Primary Effluent <1.5°C	Raw Domestic 9-10°C
Lagoon Temperature--Summer, °C --Winter, °C	12.5-20°C <1°C	14-20°C 1°C	14-20°C 1°C	14-20°C <1°C	18°C <1°C
Operation Efficiency --BOD, Summer % Removed --BOD, Winter % Removed	 73-89 41	 70 41	 36 36	 75* (61)** 59 (35.9)	 92.0 30.1
--Suspended Solids, Summer, % Removed --Suspended Solids, Winter, % Removed	92 85	-- --	-- --	-- --	76 68
--Total Coliforms, Summer, % Removed --Total Coliforms, Winter, % Removed	99.7 99.9+	-- --	73 80	99.99 97.7	-- --
Odor Conditions Near Lagoons	Noticeable at 700 ft.	Definite Problem	None Reported	None Reported	Low Level
Comments	Swamp area with brush and trees in lagoon	Nearly anaerobic with some exposed sludge banks	--	Natural lake with construction deficiencies	--
Bottom Sludge Accumulation, ft.^{3}/1000 capita/day	--	--	12.9	--	--
Information and Data Source Reference	Dawson, 1967	Dawson, 1967	Dawson, 1967		Higo, 1965

[1] This table should be interpreted as a partial listing.

*Total system efficiency at any stage

**Removal efficiency of individual cell considering only the load that particular cell receives

	Sutherland (Saskatoon), Saskatchewan Series Operation		Portage LaPrairie, Manitoba	Fort Yukon, Alaska (School)	Lazy Mountain Children's Home, Palmer, Alaska
	#1	#2			
Latitude, °N	52°	52°	52°	Arctic Circle	62°
January Average Air Temperature, °F	-1	-1	--	-12	+12
Lagoon Type	Anaerobic	Anaerobic	Facultative	Facultative	Facultative
Lagoon Depth (ft.)	17	17	--	3.0	2-5
Detention Time (days)	2.75-1.45	2.75-1.45	--	--	2+
Loading--#BOD$_5$/acre/day --#BOD$_5$/1000 ft.^{3}/day	1300-2600 3.5-7.0	-- --	30-80 --	85 (winter only) 0.75	40 0.46
Ice Conditions--Duration Starting Month to End Month --Depth (ft.)	None --	-- Periodic thin sheets	November April --	-- To bottom at edges	October April 2
Sewage--Source --Winter Temperature, °C	Raw Domestic 10-14°C	Cell #1 --	-- --	School 20°C	School --
Lagoon Temperature--Summer, °C --Winter, °C	-- 5-7.8°C	-- 2.8-4.5°C	-- --	-- <1.0°C	-- 1.5°C
Operation Efficiency --BOD, Summer % Removed --BOD, Winter % Removed	 -- --	 30-50 20-35	 85-98 36	 -- --	 -- 35-50
--Suspended Solids, Summer, % Removed --Suspended Solids, Winter, % Removed	-- --	55-70 55-70	-- --	-- --	-- --
--Total Coliforms, Summer, % Removed --Total Coliforms, Winter, % Removed	-- --	80-90 70-80	-- --	-- --	-- --
Odor Conditions Near Lagoons	Detectable at Less than 300'	Detectable at Less than 300'	None Reported	None Noticed	2 weeks in Spring
Comments	Noticed efficiency reduction w/length of service	Odors seem to be coming from floating grease, scum	Two ponds operated alternately	No overflow High groundwater area	--
Bottom Sludge Accumulation, ft.^{3}/1000 capita/day	14.0	--	--	--	--
Information and Data Source Reference	Brisbin, 1967		Sparling, 1961	Anderegg, 1960	Anderegg, 1959

Drayton Valley, Alberta (3550) Series Operation (Cell #2 out 12/20/61-4/11/62)			Stettler, Alberta (3800) Operated in Series				Norman Wells, N.W. Territories
Cell#1	Cell #2	Cell #3	Primary #1	Primary #2	Secondary #1	Secondary #2	
52°	52°	52°	52°	52°	52°	52°	65°
Edmonton +3.7	Edmonton +3.7	Edmonton +3.7	Edmonton +3.7	Edmonton +3.7	Edmonton +3.7	Edmonton +3.7	--
Facultative	Facultative	Facultative	Anaerobic to Facultative	Facultative	Facultative	Facultative	Anaerobic
5.0	5.0	4.0	10	5	5	5	2
52	33	36	5.94	6.20	75.2	87.4	23
55.5 0.255	Summer/Winter 26.9/62.0 0.124/0.285	Summer/Winter 9.2/ 38.6 0.05/0.222	1433 3.29	Summer/Winter 488 / 429 2.24 / 1.97	Summer/Winter 20.8 / 35.8 0.096 / 0.164	Summer/Winter 3.49 / 24.2 0.016 / 0.11	49 0.56
November April 2.0	November April 2.5	November April 2.5	December February Short durations 0.3	December March 0.2-1.0	November April 3.5	November April 3.5	-- --
Raw Domestic --	Primary Effluent <1.0°C	Cell #2 Effluent <0.5°C	Raw Domestic + Milk Process. 8-11°C	Cell #1 Effluent 1.5-5°C	Cell #2 Effluent 0.5-3°C	Cell #3 Effluent 0.2-2°C	Airport --
19-21°C 0.5-3°C	19-23°C 0.5-3°C	19-21°C 0.5-3°C	18-22°C 1.5-5°C	17-21°C 0.5-3°C	18-22°C 0.2-2°C	18-22°C 0.1-1°C	-- --
70.5 29.1 61.1 69.2 -- --	86.0* (52.5)** 40.8 (16.5) 56.6 (-11.5) 71.7 (8.1) -- --	92.5* (46.4)** 53.5 (21.4) 58.1 (3.4) 72.7 (3.5) -- --	37.0 28.2 69.6 64.1 -- --	63.6* (42.3)** 37.3 (12.7) 71.1 (4.9) 74.1 (27.9) -- --	93.0* (80.7)** 51.1 (22.0) 74.5 (11.8) 85.2 (42.6) -- --	95.5* (35.7)** 73.0 (44.6) 87.1 (49) 87.1 (12.8) -- --	-- 67 -- -- -- --
Low Level	Low Level	Low Level	Moderate	Moderate	Nil	Nil	None Reported
--	--	--	Algae showed in this pond from April to fall	--	Storage with release spring and fall	Storage with release spring and fall	--
--	--	--	8.75	--	--	--	--
Higo, 1965			Higo, 1965				Dawson, 1965

Camrose, Alberta (7708) Series Operation						Eielson AFB Experimental System (Fairbanks, Ak.
Primary #1	Primary #2	Secondary #1	Secondary #2	Long Detention #1	Long Detention #2	
52°	52°	52°	52°	52°	52°	64°
Edmonton +3.7	Edmonton +3.7	Edmonton +3.7	Edmonton +3.7	Edmonton +3.7	Edmonton +3.7	-10
Anaerobic	Facultative	Facultative	Facultative	Facultative	Facultative	Facultative
10.0	10.0	5.0	5.0	5.0	5.0	4.5
6.2	5.75	3.45	3.45	101	93	18
Summer/Winter 808/808 3.71/3.71	Summer/Winter 488/514 2.24/2.36	Summer/Winter 697/754 3.20/3.46	Summer/Winter 590/693 2.71/3.18	Summer/Winter 135/23.4 0.062/0.107	Summer/Winter 2.1/21.6 0.01/0.10	125 0.637
November March Nil-0.3	November March Nil-1.0	November March 2.5	November April 3.0	November April 3.0	November April 3.0	November April <1.0
Raw Domestic + Milk Process. 14-16°C	Primary #1 Effluent 1-5°C	Primary #2 Effluent 1-4°C	Secondary #1 Effluent 0.5-3°C	Secondary #2 Effluent 0.5-3°C	Long Det. #1 Effluent 0.1-2°C	Air Base Domestic 21°C
17-23°C 1-5°C	17-23°C 1-4°C	17-23°C 0.5-3°C	18-23°C 0.5-3°C	19-23°C 0.1-2°C	19-23°C 0.1-3°C	19-21°C 1°C
44.5 41.2 52.8 79.6 -- --	55.1* (19.1)** 46.6 (9.2) 56.9 (8.7) 85.8 (30.4) -- --	59.4* (9.6)** 52.4 (10.9) 73.8 (39) 87.1 (9.2) -- --	77.6* (44.7)** 52.8 (0.8) 79.0 (19.8) 91.0 (30.2) -- --	96.3* (83.5)** 60.3 (15.9) 77.7 (-6.2) 91.8 (8.9) -- --	49* (98.1)** 71.5 (28.2) 92.7 (67.3) 93.6 (21.8) -- --	73 51 87 87 -- --
Moderate to High	Moderate to High	Slight	Slight	Not Noticeable	Not Noticeable	None Reported
School Load Plus Milk Processing Plant Sludge--thick, black Offensive odor to good draining fine dark						Bottom sludge taken in Jan. & Febr. had 50% volatiles
9.5-10.1	9.5-10.1	--	--	--	--	--
Higo, 1965						Rubel, 1965

located at the Eielson Air Force Base, near Fairbanks, and operated by the Arctic Health Research Center.

The cost of construction of aerated lagoons is almost directly proportional to detention requirements; therefore, reasonable performance versus detention time prediction capabilities are necessary. Several investigations have collected data for aerated lagoon operation performance within the temperature range of 0°C to 10°C; selected sources are shown in Figure 1. The curves demonstrate that there is a high probability for obtaining BOD removal efficiencies greater than 80 percent at detentions less than 15 days with lagoons operating at temperatures less than 5°C.

Another factor affecting sizing of aerated lagoons is a volume requirement for bottom sludge accumulation. Reid (19) suggests that approximately 1½ inch of sludge accumulation should be expected when a perforated tubing aeration system is utilized for aeration and mixing.

Although several different submerged aeration systems appear completely feasible for Alaska application, some of which have been used in Canada, only the performance of perforated plastic tubing manufactured by Hinde Engineering, , Inc. has been reported in the literature. The results obtained by Reid (19), preliminary results of current Alaska Water Laboratory investigations, and aeration design approaches described by others (14,16), indicate that air delivery and placement are not nearly as critical for aerated lagoons as for extended aeration and activated sludge. Furthermore, system reliability is more important than its actual configuration for air delivery.

Although Reid (19,20,21) presents a strong case for the utilization of perforated tubing for aerated lagoons in cold regions, it is feasible to design other diffuser systems that will provide adequate mixing.

According to Van Heuvelen (31), "There are two waste treatment facilities of the diffused air tube type which have gone through several winters. The quality of effluent appeared to be satisfactory the first year, but operation and maintenance problems increased and the quality of the effluent decreased. The holes in the tubing were subject to clogging and the recommended cleaning method did not open them up. The compressors had to work under a higher pressure to overcome the additional back pressure caused by the holes being plugged. The additional maintenance caused considerable concern to the owners of these installations." In regard to cleaning procedures for the perforated plastic tubing diffuser systems, Stewart, City of Regina, Saskatchewan (28) states, "The chief mechanical problem we have encountered is due to the small holes in the aeration tubing becoming blocked. It has been necessary to lower the water levels in the lagoons to clean the tubing holes, either by cutting new holes or by deforming (bending between two rollers). It appears in our case that deforming the tubing gave more lasting results".

Black (5) reported successful operation of a Welles floating aerator at Hinton, Alberta and a Lightning fixed unit at Prince George, British Columbia in exposed temperatures of -29°C to -35°C.

Solids separation devices located in short detention primary cells (12 to 36 hour) followed by short detention aerated lagoon secondary cells (3 to 5 days) show considerable promise. Rupke (23) considers systems with the dividing wall being the solids separation system to be feasible provided adequate mixing in the primary cell is provided.

Alaska Water Laboratory
Experimental Aerated Lagoon Performance

The experience to be discussed in this section has been acquired with a research pilot facility that was constructed jointly with the Alaska Air Command at Eielson Air Force Base (EAFB), 22 miles southeast of Fairbanks, Alaska.

The climate at EAFB, located at 64 ½°N latitude, is continental including extreme temperatures of -80°F to +100°F with a mean annual temperature of approximately 25°F.

The research pilot lagoon located at EAFB was constructed with vertical walls to avoid an appropriate side-wall influence when scaling up to actual system size. The EAFB lagoon is a 20 mile PVC-lined wood crib structure, 14 feet deep, 15 feet wide, and 82 ½ feet long. Operating depth is 12 feet. The lagoon is divided into 6 cells in series. The system has been operated with a total detention of 30 days (ice influence not recalculated) and accumulative detentions of: 1.8, 4.7, 8.3, 13.7, 21.0, and 30 days. Raw domestic sewage is utilized as feed. Flow measurement is accomplished with a dump tank and a timer on the feed pump and is checked by measuring effluent flow with a V-notch weir. Periodic leak tests are conducted to further varify flow through validity. In the lagoon, each cell in series is fed from the previous cell through a 2 in. diameter, 10 ft. long hose. Commercial perforated tubing was the first aeration device evaluated. It is 3/8 in. flexible plastic tubing with 8½ slits per foot. The tubing is supported on wires 16 inches above the bottom. To alleviate sludge plugging, cell 1 has 14 tubes; cells 2 through 4 have 8 tubes; and cells 5 and 6 have 4 tubes. The second aeration system now under investigation utilizes open valve disc air diffusers. This diffuser is essentially a 2 ½ in. diameter shrouded disc, which sits on the open end of a 1/8 in. NPS pipe threaded into a header. The disc has a lift of 1/6 in. during full open air flow and "seals" the 1/8 in. pipe opening when the air is shut off.

The facility was placed in operation during the Winter of 1968-69. System efficiencies for BOD reduction are presented in Figure 2. During the winter, the temperature in the first 2 cells is a transition temperature from 20°C down to less than 5°C. The normal winter operating temperature for an aerated lagoon in Alaska varies from approximately 0° to 5°C. A close look at the family of curves in Figure 2 indicates several things. 1. There is a decided lag between winter removal curves and the summer removal curve. A small portion of this lag can be attributed to the ice influence on actual detention time, starting with cell 3. High among the potential explanations for the remaining lag are a lag in bacterial population build-up, created by low temperatures; depressed winter DO (o a high percentage of the time) in cells 1 and 2 (tube clogging); and lower endogenous respiration rates at lower temperatures. 2. Summer BOD removal efficiencies peak out between 15 and 20 days detention and drop off beyond 20 days detention. This is believed to be due to the effects of excessive algae growth. As the summer curve is for the first summer's operation and very little sludge had an

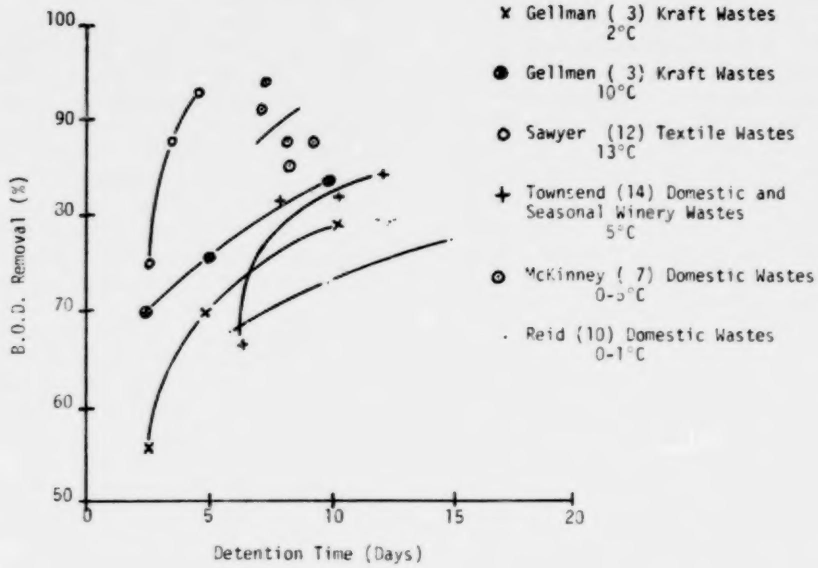


Figure 1. Aerated lagoons performance efficiencies

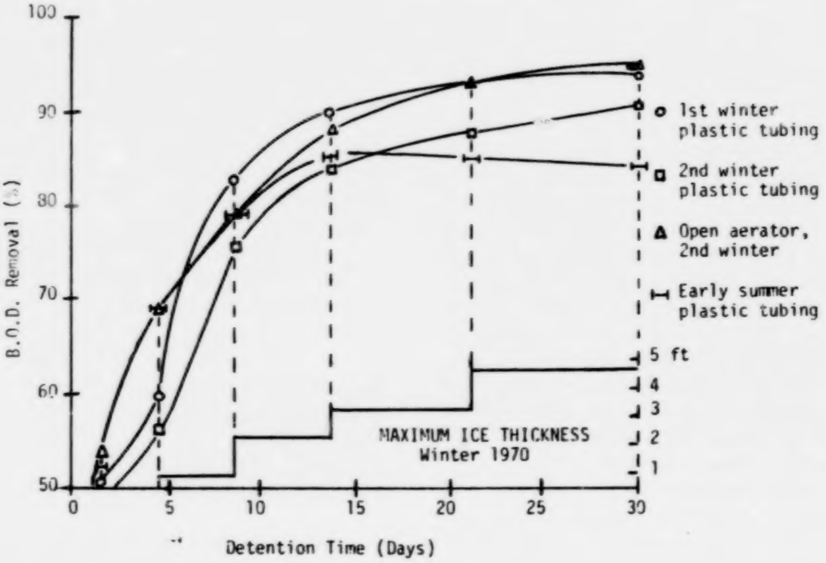


Figure 2. Alaska water laboratory aerated lagoons BOD remova efficiencies

opportunity to settle out in the last 3 cells, bottom influence is not believed to be significant in explanation of the summer efficiency reduction at higher detention. Algae were present in cells 2, 3, 4, 5 and 6 from mid-April to late September. 3. The second winter's data show a consistent BOD removal efficiency reduction of approximately 5 percent below the first winter curve when perforated tubing provided the aeration and mixing. 4. The winter BOD removal efficiency curve for valve disc open aerators shows a significant improvement over the second winter perforated tubing system, and the valve disc open aerator curve is approximately the same as the first winter perforated tubing curve. 5. Maximum ice thickness of 4½ feet to 5 feet may be expected near the effluent.

Preliminary results of tests with the system converted to a 4 cell system of the same detention time indicate that removal efficiencies for a 4 cell system are similar to those of a 6 cell system.

The total coliform removal efficiencies and effluent numbers are:

January-February	99.70%	3×10^5/100 ml
March	99.97%	2×10^4/100 ml

Contamination from the extended aeration effluent backing up during early January as a result of the common outfall, undoubtedly influenced these values. Summer 1969 effluent measurements for fecal indicators were:

	July	*August*
Total Coliforms	5.5×10^3 organisms /100ml	1.2×10^3 org. /100 ml
Fecal Coliforms	440	700
Fecal Enterococci	126	64

At the EAFB aerated lagoon, with 10½ feet submergence, 0°C, 2-10 mg/1 DO, and 80+% BOD removal, the tubing transfer efficiency calculates out to approximately 1 to 2 percent. No credit has been assumed or provided for surface transfer or BOD removal by primary sedimentation. (Ultimate BOD assumed to be 1.5 times the 5-day, 20°C BOD).

Current oprating experience, using perforated tubing at the EAFB aerated lagoon has required frequent repair and maintenance. There have been considerable tubing clogging problems which have sent more than one compressor to the graveyard. The first attempt at HC1 (gas) cleaning was successful but very shortlived, as the tubing reclogged in less than two weeks. Before gas cleaning the perforated tubing, orifice pressure drop was greater than 3 psi at an air capacity of 1.5 cu ft per minute per 100 feet of tubing. After the first dose of gaseous HC1, the air rate increased by 50% with no change in pressure drop. The compressor discharge temperature was greater than 200°F at 9 psig with ambient air temperatures of 10 to 40°F.

Northway, Alaska Aerated Lagoon Performance

Northway, Alaska is located in Interior Alaska near the Canadian border. The coldest temperature in the United States has been recorded at Northway, which is -82°F.

The Northway lagoon is a bentonite-lined wood crib structure 42 feet by 54 feet by 7½ feet operating depth. It has a longitudinal baffle, splitting it in two cells, 21 feet wide. Operation started during September 1965. Perforated plastic aeration tubing provides aeration and mixing with the spacing being according to the manufacturer's recommendations.

The Northway lagoon raw sewage averages approximately 240 mg/1 BOD. Feed temperatures vary between 2°C and 25°C. The feed flow rate of the feed pumps. The BOD removals at Northway do not vary significantly for summer or winter figures, with an average removal of 77% in the first cell and a total of 85% removal. According to the Site Manager (26), tube clogging has been a major problem, and the use of HC1 (aqueous) has not been very effective. The Northway lagoon started out as a 30 day detention system but a decreasing population has reduced the load to the present 60 to 70 day detention system.

In June 1967 sludge core samples were examined and appeared to be preserved exactly as deposited. However, in August 1967 and May 1968, bottom samples had a definite black appearance and the physical characteristics of partially digested sludge. Significant methane production occurred during the summer of 1967. The maximum depth of bottom sludge did not increase, but its location shifted between the August 1967 and May 1968 sample dates. This shifting seems to indicate that gas production resuspends bottom sludge and thus assists in providing aerobic and anaerobic digestion.

The mixing action of aerated lagoons allows the sludge zone temperature to rise above 12°C during the months of June, July, and August and part of September in Interior Alaska. Maximum bottom temperatures recorded at Northway were 19°C. Samples taken during March 1970 at Northway indicate a sludge accumulation of approximately 6 inches in the first cell and approximately 3 inches in the second cell. For 4½ years, this indicates an accumulation rate of 1 inch per year, or approximately 9 cu ft per 1000 people per year. This is less than observed rates for most stabilization ponds. It is difficult to predict what the long range accumulation and comparison will be. However, since the system is essentially a eutrophic lake, one would expect continued and accelerated bottom deposit build-up.

A Northway bottom sludge core sample taken in March 1970 from the first cell was sectioned and analyzed for fecal indicator bacteria. The 9 in. core was cut in three sections as follows: 0-3 inches bottom; 3-8 inches mid; and 8-9 inches top layer.

Organism Density

MPN counts on a per wet gram basis

No. of organisms x 103

	Top	*Mid*	*Bottom*
Presumptive Total Coliform	4.9	49	22.1
Confirmed Total Coliform	3.3	13	13
Fecal Coliform	2.4	4.9	3.3
Presumptive Enterococci	13	221	172
Confirmed Enterococci	13	221	172

It appears that the sludge located deeper in the core has a more favorable environment for preservation of fecal organisms. These bottom cores represent a stable bottom uninfluenced by resuspension dating from mid-September 1969 to 7 months of low temperature. The sludge temperature was less than 1°C and only one of four cores from the first cell at Northway had a pH less than 7.0.

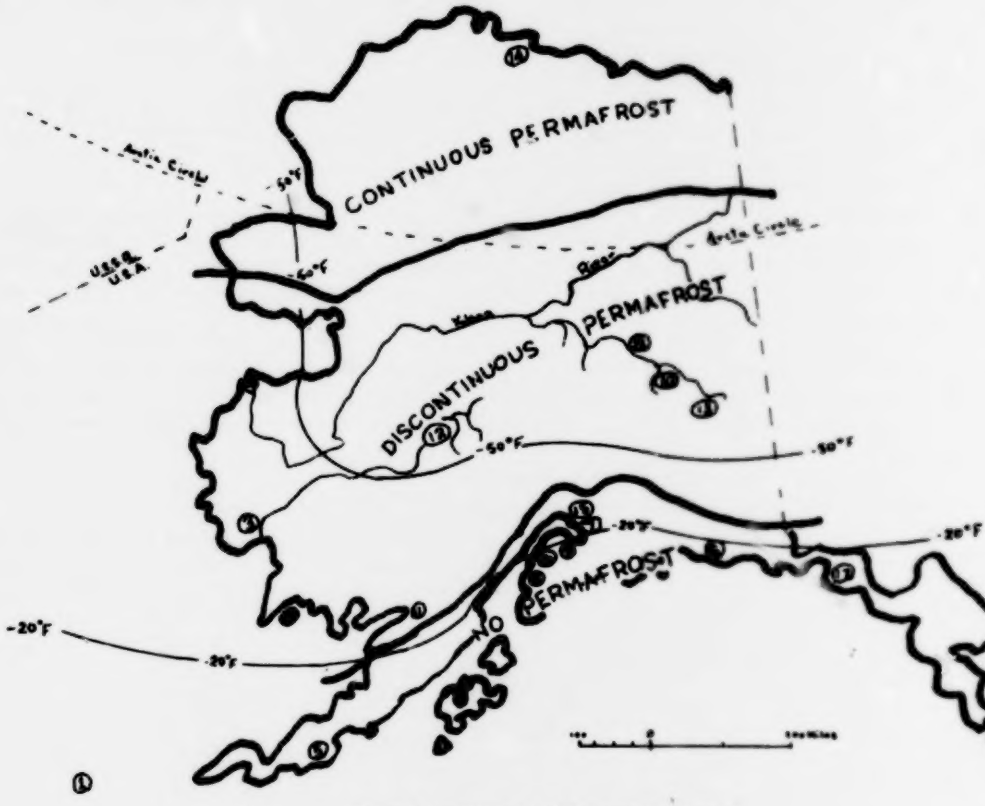


Figure 3. Locations of Alaska aerated lagoons

General Alaska Aerated Lagoon Experience

Aerated lagoons started in Alaska with the successful operation of a small unit at Eielson Air Force Base specifically placed in operation for research purposes in 1964. Reid (19,20) studied the small lagoon located at EAFB and reported BOD removal efficiencies of 68-90% and solids removal efficiencies of 67-68% for winter operation, with primary settled sewage loadings of 68-200 lbs of BOD per acre per day (0.4 to 3.1 lbs applied BOD per 1000 cu ft per day). He found that loading rates between 0.4 and 2.4 lbs applied BOD per 1000 cu ft per day gave efficiencies of 60-80% BOD removal. The system was initially a single cell system with a depth of 4.4 feet and a surface area of 0.13 acres; later this was changed to a two cell system. If we assume that the data presented by Prid (19,20) are for treatment of raw sewage, expected winter efficiencies should range between 58% at 6 days and 78% at 37 days.

There are 17 aerated lagoons in Alaska at the moment. All but one use the perforated tubing for aeration and mixing. Their locations are shown on Figure 3 and listed below.

No. on Map	*Location*	*Operating Agency*
1	Amchitka	AEC
2	Annette Island	FAA
3	Bethel	FAA
4	N. Kenai	Big Bear Oil Refinery
5	Cold Bay	FAA
6	Cordova	FAA
7	Security Squad (Elmendorf AFB)	AAC
8	Eielson AFB	AAC-FWQA
9	Fire Island (Anchorage)	FAA
10	Fort Greely	U.S.Army
11	King Salmon	AAC
12	McGrath	FAA
13	Northway	FAA
14	Prudhoe Bay	Private Constructiona
15	Scenic Park (Anchorage)	Public
16	Wildwood (Kenai)	AAC
17	Yukutat	FAA

The Alaska Water Laboratory has obtained information for seven of the aerated lagoons, which serve mainly military and FAA sites, located at Northway, McGrath, Cold Bay, Bethel, Fort Greely, Fielson Research Facility, and Fire Island. Perforated tubing was used in all cases. Performance data to establish the efficiency of these lagoons are very limited, but basically they have shown good results in the first year or so of operation.

Methods and materials of construction related to frost-heave and ice action have presented problems (26,17). Construction in permafrost should be given special consideration. The Bethel lagoon was constructed in permafrost and melted a large ice lens under one of the retaining embankments (17). The Northway lagoon was constructed with vertical 3 inch wood plank walls out of the ground and system failure is eminent (26). Its bentonite lining has proven satisfactory.

A PVC lining was used to seal the Fire Island lagoon. Its upper half has been protected by layers of hand laid rocks. No problems have been reported (18).

The Fort Greely lagoon had vertical wood partitions which were jacked up during filling in freezing weather (25). This action also destroyed the integrity of the plastic membrane liner on the bottom. An earthen embankment should also be considered an area of trouble unless there is proper selection of materials.

The clogging of perforated tube diffuser systems with resultant compressor problems seems to be a general problem (26,18,17). As clogging progresses, efficiency reduction occurs.

Fort Yukon

Facultative Lagoon performance

Fort Yukon, Alaska is located North of the Arctic Circle and has a continental climate. Although the recorded extremes do not match Northway for low or Fairbanks for high, the average annual temperature is lower than either Fairbanks or Northway and the probability of extended cold periods is greater.

The Fort Yukon faculative lagoon came into existence in 1957 under an experimental classification. Although it has an experimental classification, very little actual operating data have been collected. The Alaska Water Laboratory has initiated a study to obtain performance data for an annual cycle of operation. Results from the first field trip are presented in Figure 4 and Table 2.

The contributing population equivalent of 63 people provides a loading of 124 people per acre and is calculated as follows:

	Fraction day	*No. of people*	*Fraction yr*		*Pop.Eq.*
Student Population	½	200	180/365	=	50
Adult/Winter	1	15	270/365	=	11
Adult/Summer	1	6	95/365	=	2
					63 Total people

$$\text{Average annual sludge accumulation} = \frac{15{,}900 \text{ cu ft}}{\begin{array}{c}63 \text{ people}\\ 14 \text{ years}\end{array}} = \frac{18 \text{ cu ft}}{\begin{array}{c}\text{person}\\ \text{year}\end{array}}$$

Lagoon size: 125' x 125' x 8' deep; 3:1 slopes

Winter surface loading: (125+24)(125+24) = 22,200 sq ft

$$\frac{22{,}200}{43{,}500} = 0.51 \text{ acres}$$

$$\text{Winter loading:}\quad \frac{63}{0.51} = \frac{124 \text{ people}}{\text{acre}}; \text{ say } 100\text{-}150$$

The lagoon at Fort Yukon had an operating water depth of 8 feet in March 1970. Conversations with maintenance people at the school indicate that the lagoon depth decreases through the summer when there is no loading. The lagoon does not have an effluent, so performance figures have little meaning. The approach taken with this "leaky" lagoon has worked at Fort Yukon, as evidenced by the lack of concern by Fort Yukon residents about the lagoon existing within their community.

Conclusions

Aerated Lagoons

Aerated lagoons, when properly designed, are simple, reasonably efficient waste treatment systems suitable for small communities. Aerated lagoons, unlike facultative lagoons, do not require large surface areas, have odor problems, or have a high level of bottom sludge accumulation. Experienced operators are not required, as a properly designed system is extremely simple to operate.

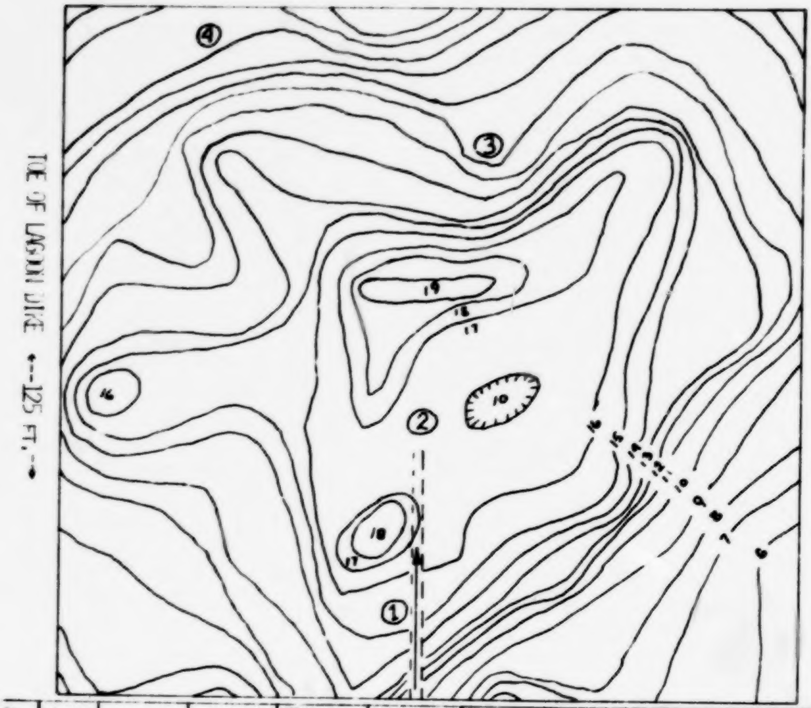
With large exposed areas, ice will form to depths of greater than 4 feet on aerated lagoons. Spring thermal expansion of ice layers may create considerable strain on embankments, especially vertical walls.

Utilizing gross bubble air diffuse systems, BOD removals of 80% to greater than 90% are feasible during the winter, depending on the number of system cells in series, individual cell detention time, and total system detention time. With perforated tubing (restricted) diffusers, BOD removals of 70 to 85% are to be expected depending on system configuration, detention time, and age of the aeration system. BOD removals greater than 86 to 88% are not feasible during the summer because of the algae load. When a restricted (partially clogged) aerator was used, the BOD removal efficiency was observed to drop as the lagoons age. The maintenance and clogging problems encountered with the restricted aeration system have not been found with the open type.

Provision of a system with a treatment efficiency (BOD removal) greater than 80% greatly increases the project's first cost (construction) but does not significantly increase operating costs.

Two lobe, positive displacement compressors will provide satisfactory performance without being enclosed in a heated building. High operating pressures greatly increase compressor maintenance problems. At 9 to 13 psig (clogged aeration system), discharge temperatures are over 200°F; at 5 to 6 psig, discharge temperatures are less than 90°F when outdoor ambient temperatures are 10-40°F.

Aerated lagoons are not efficient nutrient removal systems. On an annual basis, phosphates are removed during the summer and released during the winter, causing an apparent winter increase, but the annual net removal is



Stn. No.	Liquid / Sludge	COD	BOD	Total Solids	T. Volitile Solids	Suspend Solids	Vol. Susp Solids	pH	Coliform (Fecal)
①	L	160 mg/l	87	450 mg/l	198 mg/l	31 mg/l	20 mg/l	7.03	—
	S	20 mg/gm	—	wt % H_2O 34%	wt % Vol 3.2%	—	—	6.67	13×10^4/gm
②	L	241 mg/l	48	571 mg/l	201 mg/l	33 mg/l	11 mg/l	7.13	—
	S	10 mg/gm	—	wt % H_2O 28%	wt % Vol 1.9%	—	—	7.28	17.2×10^2/gm
③	L	219 mg/l	117	514 mg/l	156 mg/l	47 mg/l	16 mg/l	7.10	—
	S	—	—	—	—	—	—	—	—
④	L	204 mg/l	100	556 mg/l	209 mg/l	35 mg/l	12 mg/l	7.02	3.2×10^6/ml
	S	12 mg/gm	—	wt % H_2O 24%	wt % Vol 2.1%	—	—	7.09	49×10^2/gm

Figure 4. Bottom sludge depth profile (in inches) Fort Yukon Lagoon April 1970

Table 2. Enteric Indicator bacteria in sludge Fort Yukon sewage stabilization pond

Counts obtained per gram of sludge by MPN method

Sample Location (See Figure 4)

	1	2	4
Presumptive Total Coliforms*	1,300,000/gm	22,100/gm	7,900/gm
Confirmed Total Coliforms*	490,000	10,900	7,900
Fecal Coliforms*	130,000	1,720	4,900
Presumptive Enterococci*	700	240	240
Confirmed Enterococci*	310	33	49
Clostridia**	5.3×10^5	4.5×10^4	6.3×10^4

No Salmonella found in the samples**

*Dr. R. Gordon, Research Microbiologist, Alaska Water Laboratory, provided these analytical results.

**Mr. L. Miller, Chief, Bacteriology Section, Arctic Health Research Center, provided these analytical results.

essentially zero. Total nitrogen removals were approximately 30% in the summer and 0% in the winter.

Facultative Lagoons

There are four important considerations in the design and use of anaerobic and facultative lagoons for waste treatment.

Public Health Implications: Anaerobic and facultative lagoons are a definite improvement over disposal to the street, back yards, or an outdoor privy. The application of these systems to waste disposal in remote villages, where a source of reasonably priced power supply is unavailable, should be given serious consideration. Bottom sludge definitely accumulates faster than it is stabilized, so the system will have to eventually be abandoned. The question then becomes one of "how much public health hazard does an abandoned sewage lagoon create?" Deep, lightly loaded lagoons will have many years of service. Information on the degree of winter removal of coliforms is extremely scarce; however, there does appear to be a significant reduction. Pathogenic organisms have been isolated from early expeditions and camps in the Antarctic and Arctic, indicating long survival under refrigerated conditions.

Nuisance Potential: Anaerobic and facultative lagoons should always be located downwind (prevailing) and at least ¼ mile from the nearest inhabitants. An obvious and reasonably reliable means for minimizing odor production is the utilization of a design loading of 20 lbs BOD per acre per day or less. However, Alberta and the Northwest Territories have had reasonable success in minimizing odor production through utilization of deep primary cells loaded at high rates (1 to 4 day detention), followed by long detention time cells in series. The reasons why a short detention primary cell ususally will not produce large amounts of odor are not clear. It is undoubtedly a combination of factors, two of which may be high flow rates in the overlying liquids and the concentration of potential odor production in a small area.

System Loading and Process Efficiency Requirements: The evidence for utilizing anaerobic and faculitive lagoon design loads of less than 50 lbs BOD per day per acre is overwhelming. However, the favorable experience with several cells in series and utilization of a short detention time (1 to 4 days) deep primary cell is reasonably well documented, and should not be overlooked. Winter BOD removal rates of 80-90% are not feasible; therefore, serious consideration must be given to winter storage with summer treatment in a shallow lagoon, where a two cell system is feasible.

System Configuration and Construction: The influence of ice cover, frost-heave, and depth must be carefully considered in the physical design of lagoons for an extremely cold climate. Influent, cross-over, and effluent lines should be submerged, with the influent line centrally located. Canadian experience indicates that deep lagoons are preferable; however, the question exists, "what influence does the increase in depth have in the formation of a strong thermocline?" Extreme care must be exercised in application of lagoons in permafrost areas, where ice-rich soil may exist.

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A RATIONAL APPROACH TO THE DESIGN OF AERATED LAGOONS

*Edward F. Pohl***

INTRODUCTION

New Alaska State Water Quality Standards, formulated in accordance with the Federal Water Quality Act of 1965 (Public Law 89-234) and the nationwide effort to achieve pollution control, dictate a need for more efficient waste treatment facilities serving Federal establishments in Alaska. The President, by Executive Order has directed the Federal establishment to assume a leading role in this program.

An orderly program for upgrading waste treatment facilities serving military establishments in Alaska has been developed. To comply with State Water Quality Standards, the need has emerged for inexpensive treatment plants capable of producing an effluent of "secondary treatment" quality with a minimum of maintenance and operational skills. Alaskan climatic conditions require most conventional treatment systems to be heated and housed. Many of the military establishments to be served are small and isolated. Aerated lagoons specially designed for Alaskan climates have been developed as one technique for accomplishing the objectives described above.

The Air Force initiated an interservice agreement with the Artic Health Research Center, U.S. Public Health Service, College, Alaska, in 1964 to install and observe an aerated lagoon at Eielson AFB near Fairbanks. The studies conducted under the direction of Lt. Leroy Reid, used the Air-Aqua System described by Meyer (11) and Hurwitz (9) as a source of oxygen. The studies demonstrated that treatment facilities capable of meeting Air Force objectives at a cost substantially below that of more conventional facilities were technically feasible. The remaining need was to define and evaluate the various processes going on in the system in order to develop a rational basis for design.

The purpose of this study was to build a design skeleton out of the principles, as they are presently known, which could be fleshed out with additional research. The skeleton turned out to be reasonably complete. Lagoons designed in accordance with these principles have been built at Amchitka, Wildwood, Fire Island, King Salmon, Elmendorf and Fort Greely, Alaska. Performance has meet expectations at all locations.

The term aerated lagoon conventionally describes

**Chief, Sanitary and Civil Engineering Section, Alaska District, U.S. Army Corps of Engineers, Anchorage, Alaska lagoons which use mechanical or diffused aeration to avoid the problems associated with the use of algae. The flow characteristics, amount of turbulence and mixing, and efficiency of the oxygen transfer equipment determine the length of the retention time. The percentage of the total organic loading stabilized aerobically is determined by the amount of turbulence available to maintain the solids in suspension.

The aerator devices normally used in the "South 48" are surface units which achieve oxygen transfer by turbulence. They may be floating or suspended over the surface of the lagoon on a support. The spacing and horsepower of the units determine the degree of mixing obtained. Rational approaches to the design of completely mixed facilities have been outlined by Eckenfelder and O'Connor (6), Thimsen (26), and Eckenfelder (5). Digestion is assumed to be all aerobic. The relationships used are those developed for other completely mixed, 100% aerobic, activated sludge processes.

Organic loading, and thus retention time, is limited by oxygen transfer capability. As the retention time is reduced, recycle becomes necessary to return sludge washed out of the system and to maintain adequate mixed liquor volatile suspended solids (MLVSS) concentrations. A typical design for a completely mixed aerated lagoon using standard surface aerators for a typical domestic waste (BOD = 250 mg/1, k_1 = 0.23 @ 25°C) gives 70% removal with a detention time of 4.5 days (6). Recycle is not normally practiced with these units. When increased efficiencies are desired, additional cells may be provided in series with the first. The surface type aerators are not suitable for use in cold climates for several reasons. The aerator unit tends to ice up, creating maintenance problems. The high level of turbulence results in rapid dissipation of heat from the liquid body, resulting in extensive freezing. Also, recent work at the FWQA Regional Laboratory located at Fairbanks leads to the conclusion that a physical barrier may begin to form at the interface which tends to inhibit oxygen transfer at low air temperatures. The lagoons described in this paper use small diameter polyethylene tubing laid on the bottom of the lagoon in a grid pattern to supply diffused air. Two types of tubing have been used. One type has carborundum diffuser discs cemented at intervals in the tubing. This type is rigid and is supported on 6" precast concrete pedestals. The other type is formed with a lead keel on the bottom to permit it to remain upright. Tiny slits of uniform size are precut in the top to permit air to escape. In each case, the grid is connected to a blower located in a small, uninsulated, preengineered metal building on the bank of the lagoon. The resultant stream of small bubbles serves as the oxygen transfer device and imparts a gentle motion to the adjacent column of water. Partial ice cover forms over the surface of the lagoon during cold weather to conserve heat and protect the oxygen transfer device from the elements.

The pilot plant and study by Reid utilized this method of oxygen transfer. The results have been reported extensively by Reid (19) (20 (22). Based upon the results observed by Reid, the FAA has also installed similar lagoons at Northway, Bethel and Cold Bay. The results at Northway and Bethel have been monitored and reported by Reid (22). Lagoons using this type of aeration equipment have also been installed at Harvey, North Dakota; Norway, Maine; and Regina, Saskatchewan. McKinney (10) has reported on the performance of lagoons at Lee's Summit and Raytown, Missouri, which utilize the tubing described herein as a source of oxygen.

The major processes occurring in a lagoon utilizing tubing as the primary source of oxygen appear to be similar to those occurring in a conventional facultative lagoon (Fig. 1).

Figure 1. Schematic of aerated lagoon using tubing for aeration

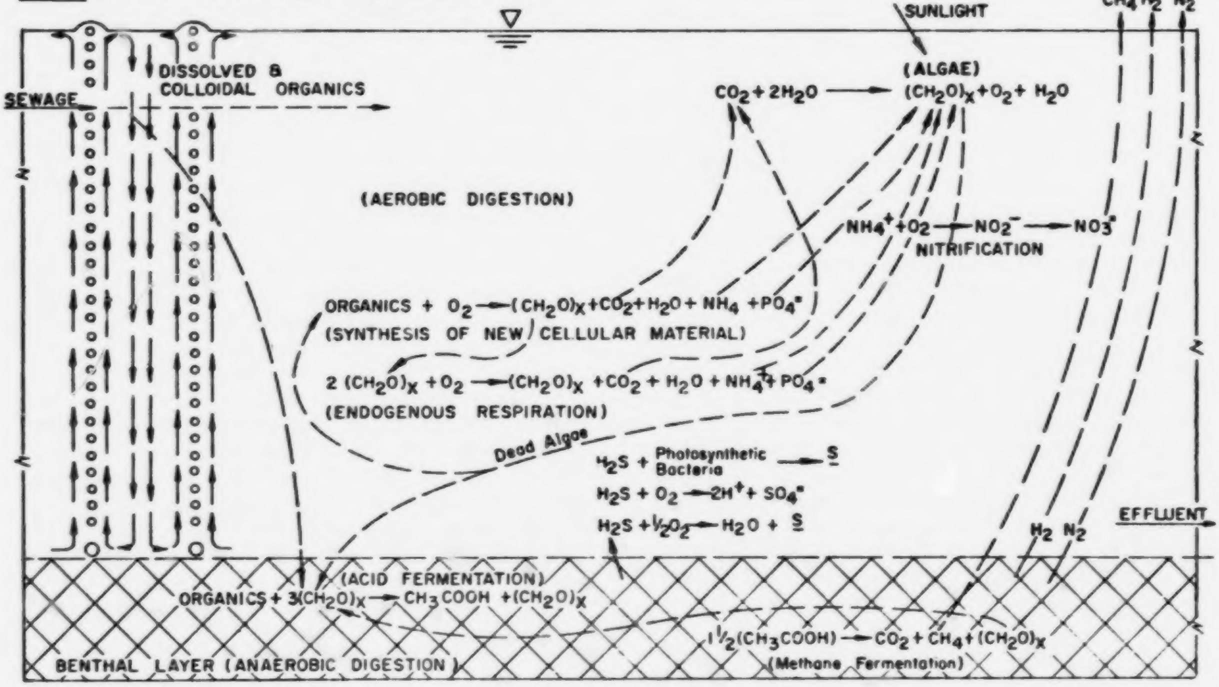
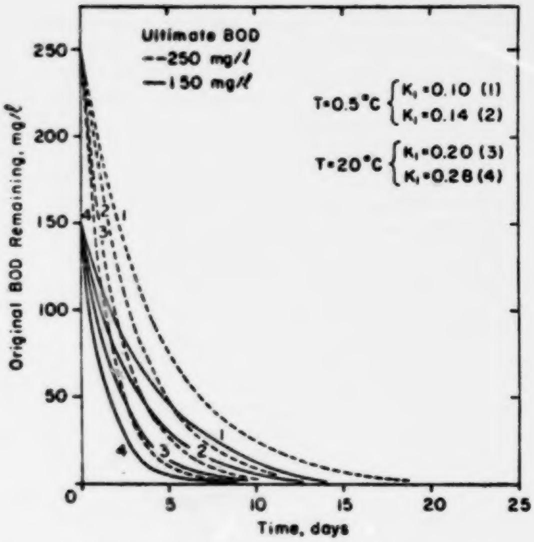

SUNLIGHT
CH_4 H_2 N_2
DISSOLVED & COLLOIDAL ORGANICS
SEWAGE
$CO_2 + 2H_2O \longrightarrow$
(ALGAE)
$(CH_2O)_x + O_2 + H_2O$
(AEROBIC DIGESTION)
$NH_4^+ + O_2 \longrightarrow NO_2^- \longrightarrow NO_3^-$
NITRIFICATION
$ORGANICS + O_2 \longrightarrow (CH_2O)_x + CO_2 + H_2O + NH_4 + PO_4^=$
(SYNTHESIS OF NEW CELLULAR MATERIAL)
$2(CH_2O)_x + O_2 \longrightarrow (CH_2O)_x + CO_2 + H_2O + NH_4^+ + PO_4^=$
(ENDOGENOUS RESPIRATION)
Dead Algae
H_2S + Photosynthetic Bacteria $\longrightarrow \underline{S}$
$H_2S + O_2 \longrightarrow 2H^+ + SO_4^=$
$H_2S + \frac{1}{2}O_2 \longrightarrow H_2O + \underline{S}$
EFFLUENT
H_2 N_2
(ACID FERMENTATION)
$ORGANICS + 3(CH_2O)_x \longrightarrow CH_3COOH + (CH_2O)_x$
$1\frac{1}{2}(CH_3COOH) \longrightarrow CO_2 + CH_4 + (CH_2O)_x$
(Methane Fermentation)
BENTHAL LAYER (ANAEROBIC DIGESTION)

Table I – Reduction in Supernatant Suspended Solids at a Depth of 12 in. Due to Sedimentation in a Quiescent Facultative Stabilization Pond. (From Oswald (15).)

Time (hr)	Suspended solids (mg/liter)	Reduction (%)
0	225	0
0.25	120	47
0.50	96	57
1.0	90	60
2.0	84	63
4.0	60	73
8.0	44	81
24.0	33	85
48.0	29	87

Figure 2. Original BOD remaining at time, t.



Settling

Air is normally introduced at the rate of from 1 to 3 ft^3 per minute per 100 ft of tubing. The resulting vertical stream of bubbles imparts a velocity to the adjacent column of water. This establishes a gentle mixing action within the zone between adjacent tubes as shown in Figure 1. The resulting velocities are reported (23) to be less than 1 ft/sec and apparently are not rapid enough to maintain all of the settleable solids in suspension. Reid (22) reports an 8- to 10-in. buildup of sludge at the inlet to the Eielson lagoon during winter 1965-1966 under heavy loading conditions. Partial clogging of the air lines occurred, which the author attributed to the weight of solids deposited over the tubing. The same phenomenon has been observed in our lagoons. This demonstrates the inability of the tubing to maintain solids in suspension immediately over the tubing. The size of material maintained in suspension between adjacent tubes would be expected to be a function of spacing. Incoming organic materials in a waste are composed of dissolved, colloidal and settleable fractions. Primary treatment of a domestic waste normally settles out approximately 40% of the total BOD indicating this comprises the settleable fraction. In a quiescent facultative lagoon 50% of the suspended solids have been observed to settle out in 15 minutes and 85% removal occurs within 24 hours (15). Much of the colloidal and soluble fractions are converted to new cellular material. Most of this remains in suspension until the tubingspacing is wide enough for it to settle. Settling rates in a quiescent pond are shown in Table I.

For a lagoon aerated with tubing, the fraction maintained in suspension and digested aerobically is larger than that in a facultative lagoon. An accurate estimate of the fraction settling into the anaerobic bottom layer will have to be determined by actual measurement. In the interim, an attempt has been made to bracket the maximum and minimum percentage of the incoming BOD expected to be digested aerobically and the resulting curves have been plotted throughout the study using both values. The values were determined as follows:

The normal domestic waste has a 5-day BOD of approximately 200 mg/l derived from a per capita contribution of 0.17 lb BOD and 100 gal. of waste per capita per day

$$\frac{0.17\ (10)^6}{8.33\ (100)} = 204 \text{ mg/l}.$$

The ultimate BOD at $k_1 = 0.20$ is 226 mg/l. Assuming one third of the incoming BOD is settled out, the fraction digested aerobically is

$$0.67\ (226) = 150 \text{ mg/l}.$$

This is a minimum value which is not likely to be attained. A slightly stronger waste (ultimate BOD = 250 mg/l), none of which settles out, was assumed for the maximum condition. For the military installations under discussion, the actual amount of incoming BOD stabilized aerobically should lie between these values.

Figure 2 is a plot of ultimate first stage BOD in its original form remaining to be stabilized aerobically vs. time for an ultimate BOD of 150 mg/l and 250 mg/l at winter temperature (0.5°C) and summer temperature (20°C). The plot is a computer solution of the conventional formula for BOD removal.

$$Y = L\ (1 - 10^{-k_1 t})$$

where

Y = BOD utilized in time t
L = ultimate first stage BOD
t = time, days
k_1 = deoxygenation constant
= reaction coefficient to the base 10
= K/2.3.

Two values for the deoxygenation constant (k_1) were assumed (0.28 and 0.20 at 20°C). Based upon work by Pohl

(18) and Carpenter, et al. (2), a value of $\theta = 1.038$ has been used for a temperature coefficient rather than the more commonly used 1.047. This gives corresponding k_1 values of 0.10 and 0.14 at 0.5°C.

Type of flow

Two types of flow, or a combination thereof, are normally encountered in a treatment process: plug flow and complete mixing. Plug flow is defined as complete mixing within an initial unit volume of liquid and zero intermixing with adjacent unit volumes as that unit volume travels the length of the container. A unit volume for this study is defined as one day's flow. The biota synthesized within a specific unit volume of fluids will tend to remain with that volume of fluid throughout its journey through the lagoon. The curves produced for BOD removal, new cellular material synthesized, and oxygen required will tend to look like those in Figures 2, 3 and 4. They will be shortened or lengthened depending upon temperature and concentration of viable organisms (by common usage assumed equal to the MLVSS).

In a completely mixed system, the concentration of BOD, MLVSS and suspended solids will be uniform throughout the vessel. The effluent concentrations will be the same as those in the container. The system will assume a single point on curves similar to those shown in Figures 2, 3 and 4 and maintain that point. This is called maintaining steady-state conditions.

Reid (21) has described the introduction of dye into the experimental lagoon at Eielson. The dye-stained liquid underwent rapid and almost complete mixing within the first cell which is his term for the area between adjacent tubes. Each cell was completely mixed before the flow moved to the next adjacent cell, where the same thing occurred. Flow in the lagoon was therefore originally thought to be true plug flow with complete mixing and zero intermixing. This, then, would have been the major distinguishing feature of lagoons aerated with tubing as opposed to the completely mixed lagoon aerated with surface type aerators. In practice, this has been found to be inaccurate. Solids and BOD concentrations at various points in the lagoon vary with the maximum concentrations at the inlet end. However, the variation has been less than 25% in every case. This indicates the flow conditions are a mixture of the two types of flow. The necessary work to define the ratio of mixed to plug flow has not been initiated at this date. It could be expected to be a function of the geometry of the basin and the design of the air lift return. The two conditions can be bracketed, however, by computing the removals based upon pure plug flow and complete mixing. Also, basins in series have no theoretical value for plug flow, but have definte theoretical value for completely mixed flow.

Completely mixed flow

The computations for completely mixed flow are as follows:

# BOD influent - # BOD effluent = # BOD removed

or $L_oQ - L_eQ = \frac{-dL}{dt} V$

Where:

L_o = ultimate influent BOD in mg/1
L_e = ultimate effluent BOD in mg/1
Q = Sewage flow in millions of pounds per day
V = Volume of lagoon in millions of pounds
$\frac{-dL}{dt}$ = monomolecular equation for the removal of BOD
$= K^1 S_a L$ for an activated sludge process functioning in the declining growth phase
$= K_1 L_e = 2.3\, k_1 L_e$
t = detention time in days
$= \frac{V}{Q}$

Then $L_o - L_e = K_1 L_e t = 2.3\, k_1 L_e t$

$$\frac{L_e}{L_o} = \frac{1}{1 + K_1 t} = \frac{1}{1 + 2.3\, k_1 t}$$

if E = percent of altimate BOD removed $= 100\,(1 - \frac{L_e}{L_o})$

then $E = 100\ \frac{K_1 t}{1 + K_1 t} = \frac{230\, k_1 t}{1 + 2.3\, k_1 t}$

and $t = \frac{E}{K_1\,(100-E)} = \frac{E}{2.3\, k_1\,(100-E)}$

If multiple cells are used and k_1 is assumed to be the same for all cells,

$E = 100\,(1 - F^n)$

Where: F = fraction of BOD in each cell
$= \frac{1}{1 + K_1 t} = \frac{1}{1 + 2.3\, k_1 t}$
n = number of cells

The above analysis of completely mixed systems ignores the BOD in the effluent contributed by biological solids. The same material balance approach used for BOD can be used to determine the concentration of biological solids for any detention time (t).

$$S_oQ + \Delta S = S_aQ$$

Where: S_o = influent suspended solids
ΔS = the net production of solids
$= a(L_o - L_e)Q - b\, S_a\, V$
$= a(L_o - L_e)Q - b\, S_a\, Q\, t$
S_a = effluent suspended solids
= solids under aeration

Substituting:

$$S_oQ + a(L_o - L_e)Q - b\, S_a\, Q\, t = S_aQ$$
$$S_o + a(L_o - L_e) - b\, S_a\, t = S_a$$
$$S_o + a(L_o - L_e) = S_a\,(1 + bt)$$
$$S_a = \frac{S_o + a(L_o - L_e)}{1 + bt}$$

This analysis produces a maximum concentration to be expected. In practice we have found that the outlet structure will act as an inefficient clarifier reducing the solids by a substantial fraction.

Plug flow

A computer program has been written by our office which makes construction of curves similar to those in Figures 2, 3, 4 and 5 a simple matter. These can then be used to determine retention times, oxygen required, etc. for the desired effluent quality.

Synthesis of new cellular material

The incoming BOD utilized is first broken down by the organisms into soluble simpler forms which can be taken through the cellular wall. The materials taken into the cells are used as building blocks for the synthesis of new cellular material, repair and maintenance of existing cellular material and for energy. The relationship is expressed by Eckenfelder (5) as:

ΔS (lb/day) = a L_r (lb/day BOD removed)
- bS_a (lb MLVSS)

Where:

ΔS = net accumulation of cellular material (which is capable of being subsequently degraded biologically)

a = the fraction of BOD removed which is synthesized to new cellular material
= 0.55 for BOD_5 or
= 0.4 for ultimate BOD (ref. 5)

L_r = the weight of incoming BOD utilized in unit time t

b = the fraction of the total cellular material oxidized per day
= the mean rate of endogenous respiration
= 0.2 (ref. 5)

S_a = the total weight of biologically active solids that is degradable at time t.

For plug flow the assumption has been made that no intermixing occurs between adjacent unit volumes in the trip through the lagoon. Thus the solids present in any unit volume at any time t will consist of unutilized incoming materials remaining in suspension, new materials synthesized, and algae. Each will consist of biodegradable and non-biodegradable fractions. The algae, as described later, will be assumed to exert no oxygen demand until they die in the fall and settle to the anaerobic bottom zone. The inert and nonbiodegradable organic fractions of the incoming and new suspended materials will accumulate in the system. Part of this will settle out and the rest will be discharged with the effluent.

The equation quoted above, as normally used, defines ΔS as the net accumulation of new cellular material. The first term (aL_r) represents the weight of new cellular material synthesized in a unit of time. This includes a small nonbiodegradable fraction (5). The second term describes the weight of the total biodegradable biomass which is destroyed by auto-oxidation in the same unit of time. S_a, then, must be that fraction of the total cellular material present which is biodegradable. However, when set up for computer solution as described below, S_a at time t will equal the $\Sigma\Delta S$ from time $t = 0$ to $t - 1$ plus aL_r for time t, and this includes the nonbiodegradable fraction which has accumulated in the system. It becomes necessary, then, to redefine ΔS as the net accumulation of cellular material which is biologically capable of being subsequently degraded through auto-oxidation. It is recognized that this interpretation will affect the constant a. Eckenfelder's values for the constants a and b were developed under conditions other than those under discussion. We intended to use them until more accurate values could be determined. The results experienced in our lagoons seem to match the results predicted by these values, and the necessary work to refine the values has not been done.

The net accumulation of cellular material using the foregoing relationships has been programmed for solution by the North Pacific Division Computer Center in Portland, Oregon. It is written in FORTRAN IV for the IBM 360/50 System which includes 225 k memory with FORTRAN IV options, level H.

$\Delta S = A - B$

A = lb solids synthesized/day
$= 8.33\ QtaL_r$
= solids concentration in a unit volume (mg/1) when Q = MGD, t = 1 day and L_r = mg/1
$= aL_r$ when $Q = \dfrac{1}{8.33}$ and $t = 1$

a = 0.4 from ref. 5

Unit volume = 1 day's flow

L_r = BOD removed from time t - 1 to time t (mg/1)
$= L(1 - 10^{-k_1 t}) - L[1 - 10^{-k_1(t-1)}]$

k_1 = 0.28 and 0.20 @ 20°C (two curves)

B = lb solids oxidized per day
$= b(\Sigma\Delta S$ for $t = 0$ to $t - 1 + A$ @ time t)

$b = b$ @ $20^\circ C\ \theta^{T-20}$
= 0.2 @ 20°C from ref. 5

$\theta = 1.038$.

The computer solutions have been plotted as ΔS vs. time (Fig. 3). The total accumulation of cellular material, including the nondegradable fraction, may be determined from empirical observations (see the section on Suspended solids in effluent).

Oxygen required for synthesis and respiration

The oxygen required per day for respiration and synthesis of new material (first stage BOD) is:

O_2/day (first stage BOD) = $A' + B'$

Where:

$A' = O_2$ required for synthesis
$= \dfrac{a'}{a} A$

$a' = 1 - a$ from ref. 5

$B' = O_2$ required for endogenous respiration
$= \dfrac{b'}{b} B$

$b' = 1.4\ b$ from ref. 5

This was added to the computer program and the results plotted (Fig. 4). From this can be determined the required oxygen transfer capability at any point in the lagoon to satisfy the first stage BOD. The cumulative oxygen demand versus time has also been plotted (Fig. 5). From this can be determined the total oxygen required.

The second stage BOD has not been considered. The critical oxygen demands (and consequently oxygen transfer capabilities) can be expected to occur in the inlet end of the pond (see Fig. 4). This is well before the onset of the second stage BOD. There is also some doubt whether nitrification occurs at all when the pond is functioning at low winter temperatures. In the summer, algae may be expected to utilize ammonia as rapidly as it is produced. Reid (22) lists nitrates in the Northway lagoon as varying from 0.00 to 0.26 mg/1 in the influent and from 0.00 to 0.32 in the effluent. The mean was 0.15 in the influent and 0.07 in the effluent.

Oxygen required by benthal layer

Accumulated organics in the anaerobic zone are decomposed to organic acids. These, in the presence of permissible pH and temperature, are converted to methane, CO_2, H_2, N_2 and other gases. According to Oswald (15) methane fermentation decreases sharply below 12°C and is almost nonexistent below 4°C. A tabulation of observed gas production from a facultative pond is given by Oswald (15).

Oswald reports that gas emission is virtually nonexistent below 4°C. This may be partially due to the

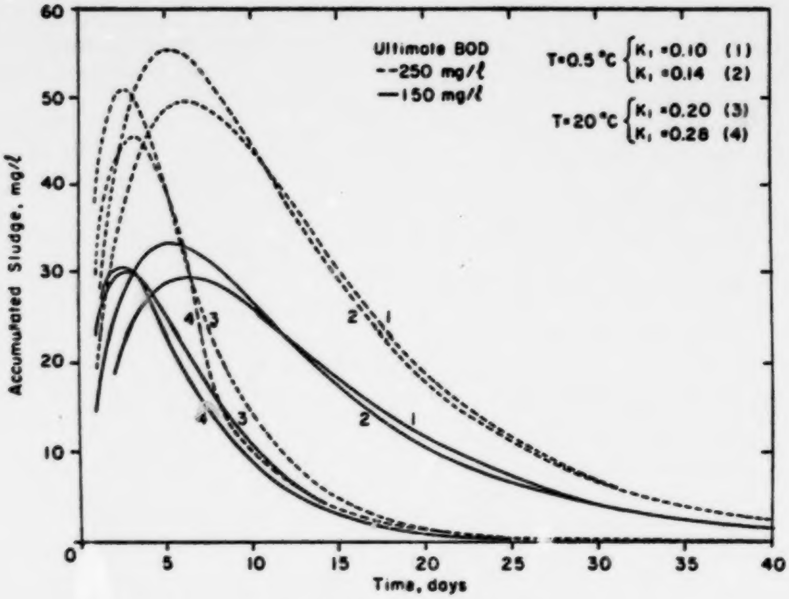


Figure 3. Net accumulated biomass.

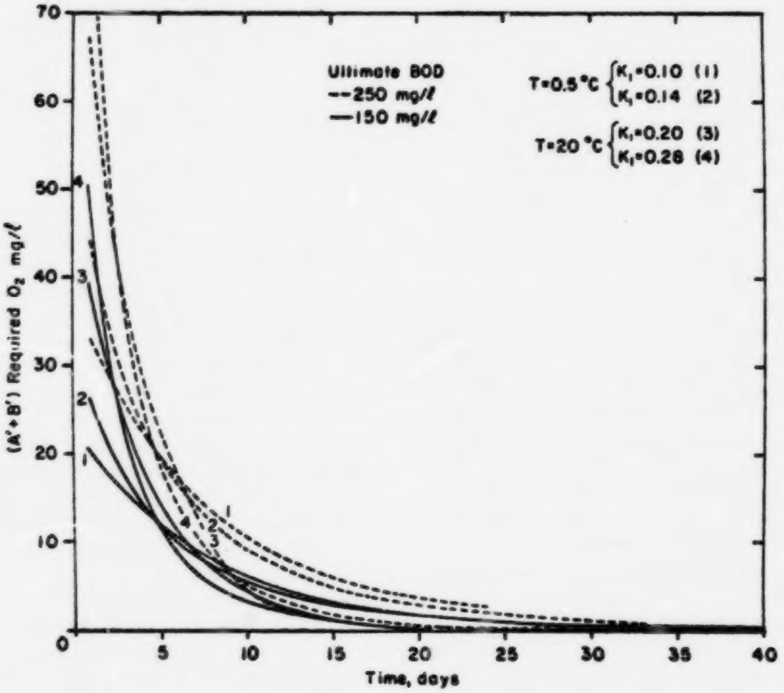


Figure 4. Oxygen required per day.

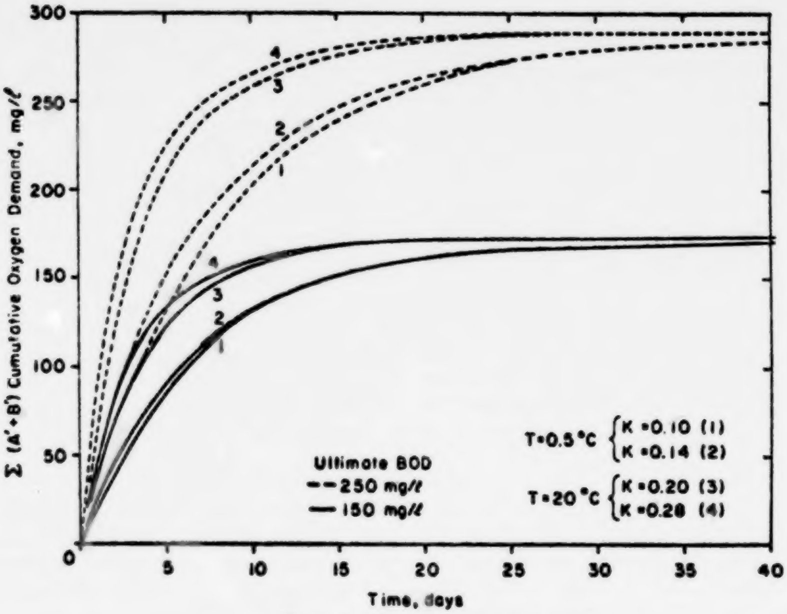


Figure 5. Cumulative oxygen demand.

increased solubility of methane and nitrogen at low temperatures. The acid formers have been observed to be slightly more active at low temperatures than the methane formers. The anaerobic zone thus functions as an unheated digester which serves as a refrigerated storehouse during winter months. Table II shows the effect of temperature on the ability of the anaerobic zone to function. The theoretical quantity of gas is a constant and is independent of temperature. It varies from 8 to 12 ft^3/lb BOD stabilized (15). Assuming an average value of 10 ft^3/lb, Table II predicts a net increase in undigested sludge will occur when the liquid temperature is below 19°C. Materials in the benthal layer will thus accumulate during the winter and be utilized during the summer. Anaerobic digestion will exert a BOD load on the zone immediately above. The demand results from oxidation of anaerobic products which diffuse up through the sludge into the aerobic zone above. Camp (1) quotes studies by Fair et al., which measured the oxygen demand of sludge layers of various thicknesses at 20°C to 25°C.

The relationship is given by

$$L_d = L_{do}\, 10^{-k_4 t}$$

Where:

L_d = total areal BOD of bottom deposits in g/m^2 after time t

L_{do} = initial value of L_d

k_4 = areal demand rate constant.

The values above are for temperatures from 20°C to 25°C. Table II can be used to relate these values to other temperatures when necessary. However, normal design in Alaska will compare summer temperatures (values given) and winter temperatures (no benthal load) with no need to interpolate. Also Table III assumes an initial deposit with subsequent fermentation. In the subject lagoons, material will be deposited continuously with a buildup in the winter and reduction during the summer. Thus, L_{do} is the value of interest. Camp makes the assumption that the oxygen demand is uniformly exerted on the overlying waters. Then for a mean sludge depth of 2.55 cm and a liquid depth of 1 m, the oxygen demand would be 227 mg/1. Consideration of the equation given above will show that long periods of time are required to reduce the oxygen demand materially. For example, for k_4 = 0.0032, almost 100 days are required to reduce the demand by 50%.

Table II – Gas Production in a Facultative Pond at a Point Where the Benthic BOD Load is 200 lb/acre day. (From Oswald (15).)

Temp (°C)	Gas collected	
	ft^3/acre day	ft^3/lb BOD*
0	0	0
4	40	0.2
8	160	0.8
12	600	3.0
16	1300	6.5
20	3150	15.7
22	4900	24.5

*Destruction of 1 lb BOD normally liberates 8-10 ft^3 gas.

Table III – O_2 Demand of Bottom Deposits. (From Camp (1).)

Mean depth (cm)	Initial vol solids (kg/m^2)	Initial areal O_2 demand		
		L_{do} (g/m^2)	Initial demand (g/m^2 day)	k_4
10.2	3.77	739	4.65	0.0027
4.75	1.38	426	3.09	0.0031
2.55	0.513	227	1.70	0.0032
1.42	0.188	142	1.08	0.0033
1.42	0.188	134	1.02	0.0033

It should be noted that the promotion of methane fermentation is highly desirable for BOD removal. Under ideal conditions, several hundred pounds of organic matter per acre day may be converted to methane (16). It is also believed that methane fermentation blocks the occurrence of sulfate reduction primarily because it controls the buildup of organic matter which is an essential for sulfate reduction. Acid fermentation without methane fermentation is undesirable, particularly in waters with a high sulfate content. This produces a condition analogous to a "stuck" digester with attendant odors if the gases are not completely oxidized in their passage through the aerobic zone. The increased production of hydrogen sulfide also obviously increases the oxygen demand on the overlying waters. Acid decomposition is more likely to be a problem in climates where summer liquid temperatures become very warm (16) – not a likely occurrence in Alaskan climates. Also, algae tend to raise the pH in the medium, preventing acid inhibition of the methane formers.

Another possibility for the digestion of bottom sludge has been reported by Reid (21). Sludge deposited around the inlet to the Eielson lagoon underwent resuspension by physical means the following spring. This may have been caused by nitrate reduction with nitrogen gas carrying the sludge to the surface. The floating sludge was then broken up by turbulence and apparently decomposed aerobically.

Murphy (12) has suggested the possible desirability of bi-level lagoons in which the aeration tubing would be suspended at mid-depth and the lagoon depth would exceed 10 ft, preferably approaching 20 ft. This would provide a quiescent, oxygen-free lower level for anaerobic digestion. Oswald (16) also suggests the desirability of designing facultative ponds in a way that produces bottom conditions favorable to methane fermentation. Such ponds appear to hold promise in Alaska for at least two reasons. The first is the relatively high efficiency which could be expected, with consequent low initial costs. Second, the shape would provide maximum protection from the elements and result in warmer liquid temperatures. Turnover caused by warming or cooling to 4°C could be expected twice a year. This could create a problem if not confined to the anaerobic levels. The idea for a deep, bi-level lagoon has not been implemented to date.

The accumulation of digested sludge has not been determined accurately, but apparently is not great. Oswald (16) says sludge accumulation in quiescent facultative lagoons at Concord, California, amounted to 1 to 2 ft^3 per annum per capita. Carryover of inert materials in lagoons using tubing for aeration should result in less accumulation on the floor of the pond than for facultative lagoons.

Influence of algae on effluent

If algae are discharged into the receiving body the effect can be predicted as follows:

1. If postchlorination is practiced to reduce coliform count, at least part of the algae may be expected to be dead. The fraction killed will depend upon the amount of chlorine used. The stoichiometric relationship is given by Oswald as:

$$C_{11}H_{29}O_7N + 14O_2 + H^+ \longrightarrow 11CO_2 + 13H_2O + NH^+_4$$

The weight of oxygen required to oxidize the dead algal material therefore is 1.56 times the weight of material oxidized. Thus the BOD = 1.56 C_e, where C_e = algal concentration.

2. If the algal cells are discharged alive into the receiving body and the receiving body has a nutrient level too low for them to grow, the cells will vegetate as long as they have access to carbon dioxide from the atmosphere and sunlight. If they vegetate, they will have a small but measurable continuous oxygen demand. Oswald and Gotaas (17) predict this as being about 1/10 of the ash-free dry weight of the cells per day at 25°C. This value would be temperature-dependent and would decrease to almost zero at 0°C. Algal concentrations from 200 to 300 mg/1 are reported to have been held for 3 months without putrefaction (5).

3. If the receiving body contains an adequate nutrient load, the cells will grow and produce oxygen. However, the inorganic ammonia, nitrates, and phosphates (termed nutrients) which result from aerobic digestion, when introduced to a receiving body, encourage eutrophication. This is an aging process which results in a high level of rooted and aquatic plant life, and can result in nuisance conditions. Algae in the treatment process, provided they can be excluded from the effluent, will strip nutrients and thus reduce the concentration introduced into the receiving body. The removal of nutrients is normally termed tertiary treatment and is the next step up the ladder above secondary treatment.

Influence of algae within the lagoon

As described above, algae will strip nutrients from the process if they can be excluded from the effluent. This, however, will result in an accumulation of algae which must in turn undergo treatment. Algal concentrations of 200 to 300 mg/1 can be expected to survive for three months without putrefaction as long as light and access to atmospheric carbon dioxide are afforded. Oswald and Gotaas (17) report that algae in the dark will remain alive for a length of time which is principally a function of temperature. Algal cells will survive in the dark for up to two weeks at temperatures near freezing, but will die and be decomposed in a few days in the dark at temperatures above 25°C. In the fall when ice cover begins to exclude light and atmospheric carbon dioxide, the algae present in the pond will die and will add to the oxygen demand in the system. At the outlet end of the lagoon where tubing is spaced far apart, part of the algae can be expected to settle to the bottom. At the cold winter temperatures they would be held in storage in the anaerobic zone to be decomposed in the spring.

Method for excluding algae from the effluent

Methods used to date have not been completely successful in excluding algae. The normal design provides a quiescent area in the vicinity of the outlet structure. Since photosynthesis is basic to the life cycle of algae, the algal concentration approaches zero when the available light is zero. Location of the drawoff point below the depth of zero light penetration should therefore eliminate algae.

Oswald and Gotaas (17) report that suspension of algal cells absorbs light within close limits in accordance with the Beer-Lambert relationship:

$$d = \frac{\log_e I_i}{C_e a}$$

Where:

d = depth at which light intensity equals zero
I_i = incident light intensity

C_e = algal concentration at the surface (mg/1)
a = specific absorption coefficient (normally 1.5×10^{-3} for sewage-grown algae).

In practice, effluent algal concentrations in the 20 to 40 mg/1 range are observed during the summer months. Other techniques for excluding algae will be necessary at those locations where these concentrations cannot be tolerated.

The concentration of algae at any point in the lagoon can be expected to be determined by the concentration of nutrients available to the algae. Assuming the nutrients are taken up by the algae as rapidly as they are produced, it should be possible to determine the algal concentration at any point in the lagoon from the stoichiometric relationships. However, in actual practice, implementation of the preceding statement is difficult. The actual algal concentration will depend upon which of the nutrients becomes limiting first. Oswald and Gotaas (17) report that a useful rule for the relationship between nitrogen and growing algae is:

$$C_c = 10N$$

Where:

C_c = maximum algal concentration (mg/1)
N = nitrogen concentration (mg/1).

the constant 10 stems from the assumption that 80% of the nitrogen in the waste is recovered and that algal cells are 8% nitrogen.

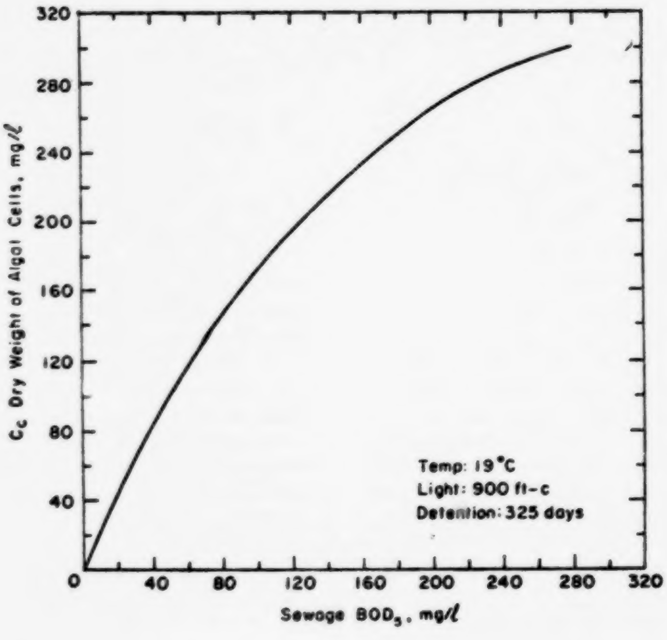
Phosphorus rarely becomes limiting because it normally does not exceed 1-1/2% of the dry weight of algae. Increased use of detergents makes it unlikely that either phosphorus or nitrogen will be limiting.

Both magnesium and potassium are also essential to algal growth. However, since algae cells normally contain approximately 0.5% potassium and 1% magnesium, and since domestic wastes normally contain more than 5 mg/1 of both of these elements, they normally will not be limiting.

Carbon is the limiting element based on the stoichiometric relationships between biological oxidation and algal synthesis. However, appreciable amounts of carbon dioxide can also be absorbed from the atmosphere at the surface. Active photosynthesis can be expected to increase pH which increases the solubility of carbon dioxide in water. Under these conditions the carbon dioxide appears in solution as the bicarbonate ion and becomes available to the algae at once. Additional research will be necessary to determine which nutrient will become limiting first. In the interim period a graph prepared by Oswald and Gotaas (17) may be used to estimate the concentration of algal cells which may be expected at different waste concentrations (Fig. 6).

Clark (3) has pointed out the potential influence of seasonal overturning on the efficiency of this method for excluding algae. For the method to be effective a quiescent

Figure 6. Effect of BOD on the concentration of algal cells in continuous cultures. (From Oswald and Gotaas (17).)



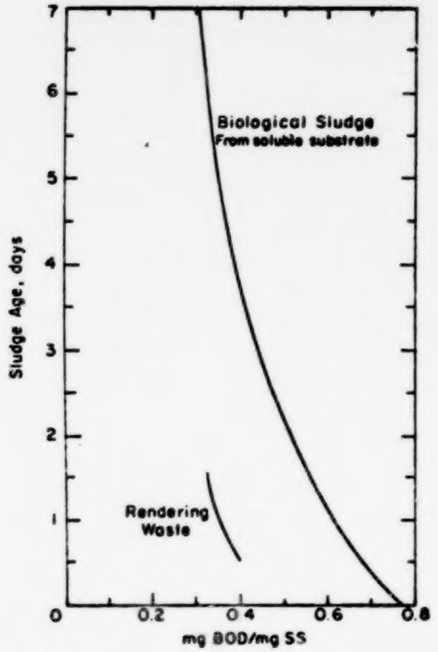


Figure 7. BOD characteristics of biological sludges. (From Eckenfelder (5).)

area must be provided. The required tubing spacing to permit algae to remain at the surface must be determined by actual observation. Reid (21) has observed no evidence of overturning with 20-ft spacing. He refers to the system described by Ogborn (13) which is used to prevent thermal stratification in freshwater reservoirs at Johnstown, Pennsylvania. Three runs of tubing 400 ft long distributing 22 ft^3/min air are sufficient to obtain complete mixing in the 75-million gallon Laurel Run Reservoir. This would indicate that overturning should not be a problem. No evidence of overturning has been observed in the seven functioning lagoons built by the Alaska District to date.

Suspended solids in effluent

The synthesis of new cellular material and subsequent autodigestion have been described under Synthesis of new cellular material. The mathematical expression defining the net concentration of cellular material after any elapsed time t has been programmed for computer solution and the results plotted (Fig. 3).

The assumption was made that endogenous respiration is a first order reaction, in which the rate of oxidation of the cellular material would be a constant percentage of the material remaining. Since the various constituents of the biomass vary in the ease with which they can be oxidized, this assumption is not strictly valid. However, pending actual determination of the change in b with time, the value used has been found to be reasonably accurate by Eckenfelder. The ΔS as defined for this study will be a measure of the remaining biodegradable material. The actual total weight of suspended solids, excluding algae, may be determined from work reported by Eckenfelder. Eckenfelder (5) has plotted mg BOD/mg SS vs. sludge age for a sludge developed from a soluble waste, and from a rendering waste which included a significant quantity of nondegradable materials (see Fig. 7). It is believed that the increasingly wide tubing spacing near the outlet end of the lagoon permits the settling out of most of the inert materials in the domestic waste under discussion. Therefore, the curve developed from the soluble substrate in Figure 7 should more accurately describe the effluent from the lagoon. The curve becomes asymptotic to a value of 0.3 as the sludge age exceeds 7 days. Then:

$$SS \text{ at time } t = \frac{\Sigma\Delta S \text{ at time } t}{m}$$

Where:

SS = Suspended solids, mg/1
ΔS = previously defined
m = value from Figure 7.

The sludge age G may be determined from the relationship

$$G = \frac{\Sigma\Delta S}{\Delta S}$$

Reduction in coliform organisms

Numerous authors have reported impressive reductions in coliform organisms in conventional facultative lagoons. Oswald (15) reports on coliform die-away observed in an experimental facultative pond at Richmond, California (Fig. 8). He also describes a relationship recently published by Marais and Shaw in which a rational method is presented for mathematical prediction for coliform removal. However, the method does not include a correction for temperature, and was shown by Oswald to be incorrect at temperatures other than those used by Marais and Shaw. Oswald and Gotaas (17) state that in laboratory and pilot plant tests they were able to observe no reduction in coliform organisms other than the normal die-away, except in certain light-saturated cultures. These were affected by the known germicidal effect of natural sunlight.

Camp (1) describes the rate of decrease in coliform count in natural waters as being mathematically

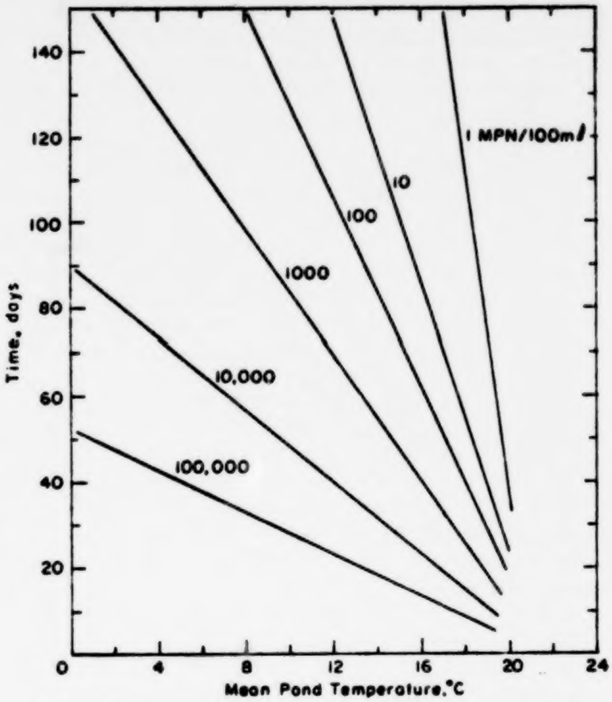


Figure 8. Observed MPN/100 ml in a single facultative pond as a function of time and temperature. Initial MPN 10^8/100 ml (from Oswald (15).)

$$\frac{N_T}{N_O} = 10^{-K_b t}$$

Where:

N_T = count after time t, days
N_O = initial count
K_b = bacterial decrease rate constant (see Fig. 9).

Depth

Lagoons which depend on algae and surface reaeration for oxygen obtain their oxygen supply at the surface. Thus the ratio of surface area to total volume becomes critical and depths are limited to 3 to 5 ft for facultative lagoons and 1 ft or less for high rate aerobic ponds. Tubing is the prime source of oxygen for the lagoons discussed herein. Oxygen transfer is accomplished at the bubble air/water interface. The efficiency is a function of bubble size and duration of contact at any given temperature and initial oxygen concentration. Thus, an increase in depth will increase the oxygen transfer efficiency because it increases the duration of contact at the interface. Increase in depth has several other advantages. Less land is required for a given detention time. Less surface area is offered for the growth of algae if algae must be excluded from the receiving body. Finally, reduction in surface area will result in less heat loss at the surface.

The decreased metabolic rates, shown graphically in Figures 2 and 3, will normally make the winter detention time critical for sizing the lagoon. The volume expected to be occupied by ice should be deducted from the total volume available when computing detention time. The depth selected will depend upon the character of the site selected, location of groundwater and nature of the material. Depths exceeding 10 ft probably will require multistage blowers. The blower selected must be capable of producing oil-free air.

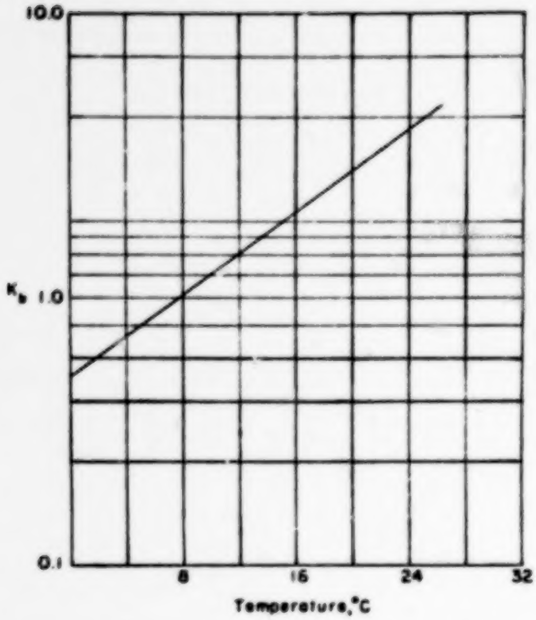


Figure 9. Effect of water temperature on values of K_b for death rate of coliform organisms. (From Camp (1).)

Oxygen sources

The demand for oxygen at any point in the lagoon will be the demand for synthesis and respiration (Fig. 4) plus that exerted by the benthal layer. During the summer, it must be satisfied by the tubing, surface reaeration and algae. Winter ice cover will reduce these sources to the tubing. Algae can be expected to supply up to 50 lb/acre day under ideal summer conditions (16).

Extensive work has been reported by Dobbins and others on the rate of absorption of oxygen from the atmosphere at the surface (normally termed surface renewal). The parameters are a function of temperature, turbulence, percentage saturation at the surface, wind velocity and liquid depth. Camp (1) presents an empirical method for evaluating these parameters. Oswald (16) reports that less than 40 lb oxygen/acre day may be introduced into a quiescent lagoon even under anaerobic conditions. The presence of algae at the surface can easily produce saturated conditions which will effectively prevent oxygen transfer. Therefore, it probably is best to ignore this source of oxygen as a design parameter pending further evaluation.

An evaluation of oxygen transfer by the tubing would normally be accomplished using the two-film theory which assumes liquid and gas films at the interface which constitute the barriers to oxygen transfer. Using Fick's law it can be shown that

$$dm = K_L A(C_s - C)$$

where

$\frac{dm}{dt}$ = time rate of mass transfer by diffusion

K_L = liquid film coefficient

A = total cross-sectional area through which diffusion occurs

C_s = oxygen saturation concentration

C = oxygen concentration in the medium

or

$C_s - C$ = dissolved oxygen deficit = concentration gradient.

The form most often used is:

$$dc = K_La(C_s - C)$$

where

a = ratio of interfacial area to liquid volume

K_La = overall mass transfer coefficient.

Eckenfelder (4) has shown that for bubble aeration, oxygen transfer characteristics can be correlated according to the Sherwood, Reynolds and Schmidt numbers.

Oxygen transfer efficiencies have been determined for the two types of tubing used by industrial laboratories under contract with the manufacturers of the tubing. The oxygen transfer efficiencies were reported for different depths of distilled water at 20°C for tubing delivering 1 and 2 ft^3/min of air per 100 ft of tubing.

$$\text{The percent efficiency} = \frac{\text{wt } O_2 \text{ absorbed/unit time} \times 100}{\text{wt } O_2 \text{ supplied/unit time}}$$

These efficiencies were determined using distilled water. The K_La for a waste will be influenced by the materials dissolved and in suspension, and will differ from that for distilled water. The relationship is given by Thimsen (26) as

$$\frac{K_La \text{ (waste)}}{K_La \text{ (water)}} = \alpha$$

He reports α as varying from 0.5 to 1.0 depending upon the type of waste and the method of aeration. This value should be determined by investigation. In the interim, it is suggested that the low value (0.5) be used.

It should be noted that biological activity will be aerobic if the dissolved oxygen concentration exceeds 0.5 mg/1. Higher dissolved oxygen concentrations may be useful to supplement the supply of oxygen in the receiving body. The nature of the receiving body will determine the value of designing for a high concentration in the effluent. Until the design parameters described herein are more accurately evaluated, it may be dangerous to design for a value of less than 50% of saturation.

SUMMARY

A rational approach to the design of aerated lagoons using tubing as the source of oxygen has been presented. [illegible] various parameters have been identified and an evaluation of each attempted based upon present knowledge. An attempt has also been made to identify areas where additional research is needed to accurately evaluate the parameters.

A typical design should evolve in three stages. Each stage should be evaluated for winter and summer operation.

1. The first stage should be a joint evaluation of the receiving body with the agency responsible for protection of the quality of the receiving body to determine the required effluent quality. For domestic wastes emanating from a military installation, this would normally be a determination of allowable BOD, suspended solids and coliform organism count in the effluent.

2. The second stage would be the determination of the required detention time to accomplish the objectives developed in stage 1. Effluent BOD will be a combination of the items described under the sections on Settling, Completely mixed flow, Plug flow, Synthesis of new cellular material and Influence of algae on effluent. The coliform count can be determined from the section Reduction in coliform organisms. The effluent suspended solids can theoretically be determined from the section Suspended solids in the effluent. In practice a major fraction of the solids will settle out in the pond and will usually be 50% or less of the value so determined.

3. The last stage will be an evaluation of the oxygen demand within the lagoon to determine tubing quantity and spacing and total air requirements. Total oxygen requirements would be the sum of the sections on Oxygen required for synthesis and respiration (Fig. 5) and Oxygen required by benthal layer. The distribution of the oxygen transfer equipment can be determined from the same two sections and from Figure 5.

The constants used for actual design should be the conservative values until they can be more accurately defined. The resulting design should be conservative. It is anticipated that research findings will modify and more sharply define this design approach with resulting increased treatment plant efficiency.

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AERATED LAGOONS FOR POTATO PROCESSING WASTES

*Kenneth A. Dostal***

INTRODUCTION

Production of potatoes in the United States has increased from 19.5 billion pounds per year in 1951 to about 29.5 billion pounds per year in 1968.(1) The four western states of Idaho, Washington, California, and Oregon accounted for most of this increase. In fact, from 1958 through 1968 the production by these four states increased 5.1 billion pounds; almost twice the increase of 2.6 billion pounds for the whole United States.

A considerable amount of this increased production can be attributed to the increased processing of specialty items such as french fries, hash browns, etc. The quantity of frozen potato products has increased from 129 million pounds in 1955 to about 1.5 billion pounds in 1966. A ten-year projection made by DuPont(2) predicted an increase of 170 percent to nearly 4 billion pounds in frozen products from 1966 to 1976 with an expected population increase of about 9 percent for the same period.

Processing of a ton of potatoes results in wastes with a population equivalent of about 500. Thus, the current waste load generated by processing plants is equivalent to about 10 million people. Even if all processors had primary treatment, which they do not, the discharged effluents would be equivalent to nearly 5 million people. Nearly one-fourth of this quantity is discharged in the State of Idaho, thus it is apparent why the Potato Processors of Idaho Association has been working on various studies on secondary treatment of potato processing wastes.

In 1964 the consulting firm of Cornell, Howland, Hayes & Merryfield was retained by the Potato Processors of Idaho Association to assist in the design and operation of secondary treatment, pilot plant facilities. As a result of this work(3), design criteria were established for secondary treatment by activated sludge, trickling filter and conventional stabilization ponds. One of the conclusions of this work was: "Additional study should be given to anaerobic ponds and flow-through aeration basins to determine more closely the capabilities of these systems in secondary treatment of potato processing wastes". This, coupled with the fact that both trickling filter and activated sludge treatment plants have high capital and operating

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costs, prompted the initiation of the study reported herein.

In July 1966 a request to the regional office of FWQA was received from the Idaho State Health Department for technical assistance in the development of secondary treatment methods for potato processing wastes. A memorandum of understanding was signed by the three participants: Potato Processors of Idaho, Idaho State Health Department, and FWQA.

The objectives of this study were to investigate two possible methods of secondary treatment:

(1) A surface-aerated aerobic lagoon, and

(2) An anaerobic lagoon followed in series by a surface-aerated aerobic lagoon.

DESCRIPTION OF FACILITIES

The pilot plant facilities were located at the site of a processor's primary waste treatment plant in Burley, Idaho. This treatment plant receives the wastes from three potato processing plants which process over 1,000 tons of potatoes per day. Two of the plants account for 87 percent of this weight and produce french fries, flakes, granules, and other potato specialties. The remaining 13 percent of the potato tonnage is processed in a starch plant.

A flow diagram of the processor's full-scale primary waste treatment plant is shown on Figure 1. Waste streams from the three processing plants are combined in a receiving tank prior to passage through two 5-foot diameter, 10-foot long rotary drum screens. Solids retained on the 20-mesh screens are removed and stored in bins. The screened waste then enters a 100-foot diameter primary clarifier which operates at an overflow rate of about 800 gallons per day per square foot. Effluent passes through a Parshall flume prior to discharge to the Snake River. Sludge from the primary clarifier is thickened in a centrifuge and also stored in bins. Both the screenings and the thickened sludge are trucked to cattle feedlots and used for feed.

Figure 2 shows a flow diagram of the pilot plant facilities used for this study. Three earthen ponds lined with gunite were used. Each pond was 40-feet square at the water surface and approximately 10-feet deep which resulted in a volume of about 50,000 gallons each. Primary clarifier effluent was pumped to aerobic pond I and anaerobic pond II. Overflow from pond II was pumped to aerobic pond III.

Pond I contained a 10-horsepower fixed aerator. The second pond was covered with three-inch thick styrofoam blocks to help retard heat loss and control odors. A pump was installed to recirculate the contents from the bottom to the top so that the anaerobic pond would be completely mixed. Pond III contained a 5 horsepower floating aerator. Sample locations were as follow:

1. Primary clarifier influent after screening,
2. Primary clarifier effluent,
3. Pond I effluent,
4. Pond II effluent, and
5. Pond III effluent.

OPERATION

The pilot plants were operated for two processing seasons of 9 to 10 months each. During the first season, pond I was not in operation because of late arrival of the 10 horsepower aerator. The hydraulic loadings were varied over a wide range so that a spectrum in results would be obtained. Feed rates in most of the runs were in the 3 to 15 gallons per minute range which resulted in theoretical detention times of 2 to 10 days. When all three ponds were in operation, the feed rate to ponds II and III, which were in series, was twice the feed rate to pond I. This resulted in equal over-all detention times through the two systems.

As mentioned earlier, the facilities were sampled routinely in five locations. Hourly grab samples were taken for eight hours and composited. In general, analyses were performed according to Standard Methods[4] and included pH, alkalinity, temperature, dissolved oxygen, settleable solids, BOD, COD, organic carbon, ammonia, nitrite, nitrate, Kjeldahl nitrogen, total and orthophosphate, as well as total, suspended, total volatile, and volatile suspended solids. During the second season, soluble COD, BOD, and organic carbon were also run.

RESULTS

Table 1 shows the average quantities of waste constituents generated per ton of potatoes processed. These figures are averages for the two-year study and are based upon the concentrations measured in the primary clarifier influent – after passage through the two 20-mesh screens. As shown, 4,200 gallons of fresh water were used per ton of raw potatoes and the waste stream contained 90 pounds of BOD, 210 pounds of COD, 110 pounds of SS, 0.6 pounds of total phosphate as P, and 3.5 pounds of total nitrogen as N.

Table 2 presents some of the results on the primary clarifier influent and effluent. Since both main processing plants used caustic peeling, the waste had a high pH. The influent had a median of 11.3 and effluent 10.4. Temperature drop across the clarifier averaged 4°F from 78 to 74. It is readily apparent that the BOD, COD, and SS concentrations varied considerably. Influent BOD averaged 2,730 mg/l with a standard deviation of ±790 mg/l. The effluent averaged 1,600 mg/l for a reduction of 41 percent. The COD was reduced by 45 percent, and the SS by 73 percent. In addition, total nitrogen and total phosphates were reduced by 21 percent.

The effluent from the clarifier which was the feed to the lagoons had an average BOD:N:P ratio of 100:5.2:0.9 although this ratio varied markedly from one day to the next. This is almost the same as the often quoted requirement of 100:5:1 for aerobic biological treatment.

A summary of the data collected on aerobic pond I is shown in Table 3. The three runs with average detention times of 7.8, 4.2, and 10.4 days had average BOD loadings of 11, 24, and 10 lbs/day/1000 cu.ft., respectively. Reduction in total BOD, that is with the suspended solids in the effluent, ranged from 55 to 82 percent. Reduction of COD across the pond ranged from 31 to 61 percent. Effluent suspended solids ranged from 630 to 700 mg/l, thus, they contributed considerable BOD and COD. This is shown by the column under BOD_s which is based on centrifuged effluent samples. Reduction in soluble BOD ranged from 76 to 95 percent.

Table 4 presents some of the data collected on anaerobic pond II. Since most of the runs were made at relatively high hydraulic rates and the contents were completely-mixed, percentage organic reductions were very low. The primary purpose of this pond was not BOD

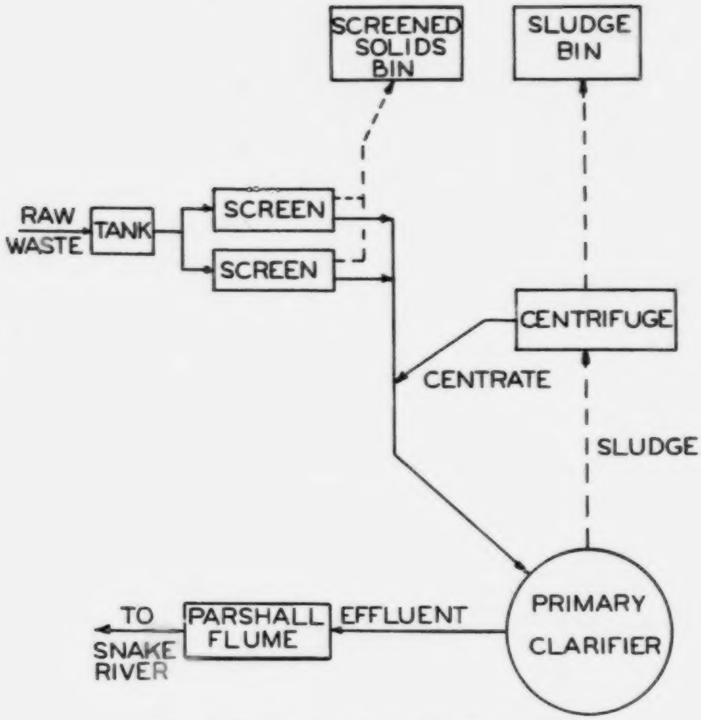
SCREENED SOLIDS BIN
SLUDGE BIN
RAW WASTE
TANK
SCREEN
SCREEN
CENTRIFUGE
CENTRATE
SLUDGE
PRIMARY CLARIFIER
EFFLUENT
PARSHALL FLUME
TO SNAKE RIVER

Figure 1. Flow diagram of primary treatment plant

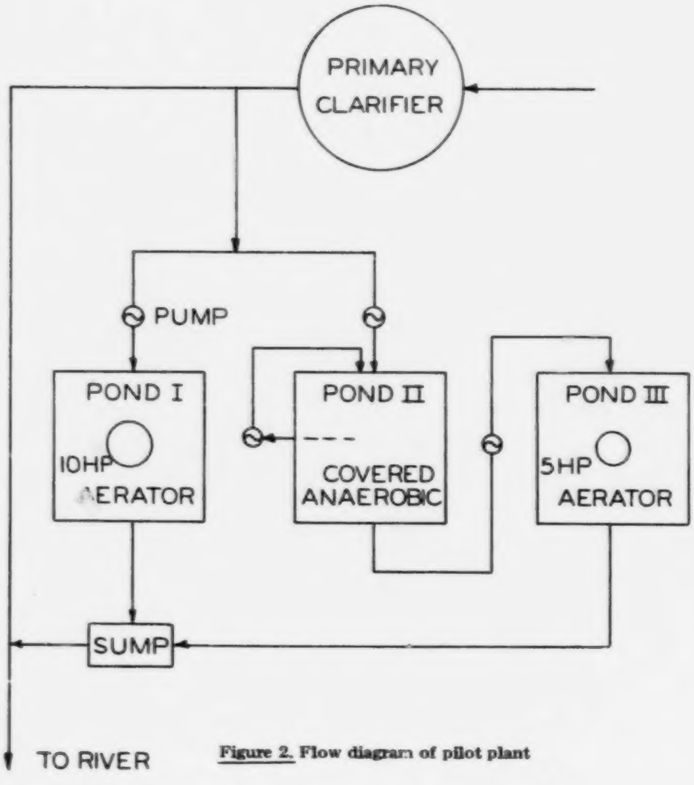
PRIMARY CLARIFIER
PUMP
POND I
10HP AERATOR
POND II
COVERED ANAEROBIC
POND III
5HP AERATOR
SUMP
TO RIVER

Figure 2. Flow diagram of pilot plant

Table 1 — Waste Production Per Ton of Potatoes Processed

Parameter	Quantity*
Quantity, gal.	4200
BOD, lbs	90
COD, lbs	210
SS, lbs	110
T-PO_4 as P, lbs	0.6
T-Nit. as N, lbs	3.5

*Measured after screening

Table 2 — Operational Characteristics of Primary Clarifier

		Clarifier Influent	Clarifier Effluent	Remova %
pH	Range	10.6-12.1	7.1-11.6	
	Median	11.3	10.4	--
Temp. °F	Range	68-82	63-79	
	Median	78	74	--
	Std. Deviation	±3	±3	
BOD mg/l	Range	1220-5150	650-2570	
	Mean	2730	1600	41
	Std. Deviation	±790	±390	
COD mg/l	Range	3120-9860	1440-6000	
	Mean	5390	2960	45
	Std. Deviation	±1520	±750	
SS mg/l	Range	280-5900	80-3190	
	Mean	2320	620	73
	Std. Deviation	±1150	±490	
TKN as N mg/l	Range	73-225	38-175	
	Mean	107	84	21
	Std. Deviation	±28	±25	
TPO_4 as P mg/l	Range	7.2-45.1	6.0-37.2	
	Mean	18.0	14.2	21
	Std. Deviation	±6.9	±6.1	

Table 3 — Data Summary for Aerobic Pond I

Run No.	D.T. days	Temp. °F	COD*		SS*		VSS*		BOD*		BOD_5*	
			In	Out	In	Out	In	Out	In	Out	In	Out
1	7.8	44	2210	860	490	630	440	560	1410	270	1060	70
2	4.2	44	2160	1480	470	700	420	610	1580	710	1440	340
3	10.4	47	2390	1190	310	650	280	620	1700	310	1440	75

Run No.	LOADINGS - lbs BOD/day			Reduction - %		
	Acre	1000 cu.ft.	lb. MLVSS	COD	BOD	BOD_5
1	1840	11	0.32	61	81	93
2	3870	24	0.62	31	55	76
3	1660	10	0.26	50	82	95

*mg/l

Table 4 — Data Summary for Anaerobic Pond II

D.T. days	Temp. °F	lbs BOD/day/ 1000 cu.ft.	BOD		BOD Red. %	COD		COD Red. %
			In	Out		In	Out	
30	54	3.3	1550	780	50	3110	1640	47
8.8	61	11	1460	1180	19	3950	2510	36
5.1	68	22	1770	1600	10	3280	2930	11
2.4	71	46	1690	1520	10	3400	2880	15
7.8	62	12	1470	1240	16	2190	1570	28
2.1	63	45	1500	1460	3	2210	1770	20
3.1	62	34	1690	1610	5	2280	2170	5
5.2	68	19	1610	1410	12	2390	2000	16

Table 5 — Data Summary for Aerobic Pond III

Run No.	D.T. days	Temp. °F	COD		SS		VSS		BOD		BOD_5	
			In	Out	In	Out	In	Out	In	Out	In	Out
1	8.8	39	2380	1400	250	730	240	630	1150	180		
2	5.1	44	2930	1250	490	820	420	790	1600	270		
3	2.4	56	2910	2160	260	860	210	710	1510	640		
4	3.9	44	1540	860	410	580	360	510	1150	380	980	80
5	2.1	43	1770	1600	400	810	360	650	1460	730	1350	485
6	3.1	43	2170	1300	470	890	410	790	1670	550	1470	190
7	5.2	50	2150	1100	410	510	340	500	1480	370	1440	110

Run No.	LOADINGS - lbs BOD/day			Reduction - %		
	Acre	1000 cu.ft.	lbs. MLVSS	COD	BOD	BOD_5
1	1500	8	0.21	41	84	
2	3660	20	0.40	57	83	
3	7400	40	0.88	26	58	
4	3000	18	0.58	44	67	92
5	7150	44	0.96	10	50	64
6	5460	33	0.68	40	67	87
7	2900	18	0.57	49	75	92

Table 6 — Values for Constants

Constant	Values* Pond I	 Pond III
a	0.63	0.63
b	0.06	0.06
k	0.64	1.1
Ks	110.	140.
c	$\frac{1.7-\log(t)}{2.5}$	$\frac{1.7-\log(t)}{2.5}$

*for temp ~ 45°F

reduction, but to see if the molecular structure of the waste constituents could be altered so that they would be degraded faster aerobically in pond III. As shown, most of the runs were made at detention times less than 9 days with BOD loadings in the 11 to 46 lbs/day/1000 ft^3 range. For these runs, the BOD reduction averaged less than 20 percent.

A summary of the data collected from pond III is shown in Table 5. The hydraulic detention time ranged from 8.8 days to 2.1 days. As the load was increased from 8 to 44 lbs BOD/1000 cu ft./day, the BOD reduction decreased from 84 to 50 percent. Soluble BOD reduction for the last four runs ranged from 64 to 92 percent at BOD loadings of 18 to 44 lbs/1000 cu.ft./day.

DISCUSSION

In order to make the data collected on the three ponds useful to others, it was desirable to use existing equations and develop the necessary constants to that the kinetics could be adequately described.

The following three equations(5) were used to calculate the effluent soluble BOD, MLVSS and total BOD, respectively, from the two aerated cells:

$$s = \frac{Ks(1+bt)}{akt-(1+bt)} \qquad (1)$$

$$M = \frac{(Fo-s)(Ks+s)}{kst} \qquad (2)$$

$$BOD_T = s + c\,M \qquad (3)$$

where:

F_o = total influent BOD, mg/1
BOD_T = total effluent BOD, mg/1
s = soluble effluent BOD, mg/1
M = MLVSS or effluent VSS, mg/1
t = detention time, days
a = growth yield constant
b = microorganism decay constant, day^{-1}
k = maximum rate of BOD utilization per unit weight of microorganism occurring at high BOD concentration, day^{-1}
K_s = half velocity constant equal to the BOD concentration when the rate of BOD utilization is equal to one-half the maximum rate, mg/1
c = BOD equivalent of the effluent VSS

The values derived from the pilot plant data for the various constants are tabulated in Table 6. As shown, the values for the growth yield constant, "a", and the microorganism decay constant, "b" were the same for both aerobic ponds. As others have observed, the value of "c" varied with sludge age or for this study, hydraulic detention time. The main difference between the two aerated cells was in the value of "k", the maximum rate of BOD utilization. The value of 1.1 for pond III was 70 percent greater than the value of 0.64 for pond I.

Although the percentage BOD reduction across anaerobic pond II was rather small, the pond apparently altered the molecular structure of the more complex organic materials such that they were degraded more rapidly aerobically in pond III. Additional benefits could also result from the use of an anaerobic pond preceding an aerobic pond in a full-scale installation. First of all, without mixing in the anaerobic pond, the effective MLVSS in the anaerobic pond would build up with time and the BOD reductions across the pond would increase accordingly. Secondly, anaerobic-aerobic ponds in series with the anaerobic pond covered would have less temperature drop than an aerobic pond with a detention time equal to that of the two ponds in series. This would increase the rate of BOD utilization in the combined anaerobic-aerobic ponds over than in the straight aerobic system.

The rate of BOD utilization in anaerobic pond II was best described by the following equation:

$$BOD_T = Fo-34\,t\,(1.04)^{T-20} \qquad (4)$$

where:

T = temperature, °C

Since only the total influent BOD was needed for aerobic pond III, no attempt was made at developing equations to predict the soluble BOD or VSS in the effluent from anaerobic pond II.

The final report(6) on this study compares the measured BOD and SS effluent values with those calculated using the four equations and constants shown above. In general, very good agreement was obtained – in most cases – within the accuracy limits of the analytical tests.

CONCLUSIONS

Conclusions drawn from this two-year study are as follow:

1. Over 90 percent BOD reduction can be obtained by primary clarification, plus either an aerobic lagoon or anaerobic-aerobic lagoons in series.
2. Both systems are economically feasible, the choice will depend upon local costs and conditions.
3. The BOD reduction across the ponds can be adequately described by available models for which the necessary constants were derived from this study.
4. Chemical additions were not necessary for either pH control or inorganic nutrient adjustment.
5. A method of removal of suspended solids from the effluent will probably be required for most applications.

FULL-SCALE FACILITIES

Since this study was reported nearly a year ago, it is of interest to note what use, if any, has been made of the information generated. At least three full-scale systems have been designed and built as a result, at least in part, of the work reported above. Gray and Osborne, consulting engineers in Yakima, Washington, have recently completed two installations on potato processing wastes. Both locations have primary treatment followed by complete-mixed aerated lagoons. One is located in Quincy, Washington, and it had a design load of about 28,000 lbs BOD/day with a flow of 1.8 mgd. The aerated lagoon contains four 150 horsepower aerators and provides 9.6 days detention time. It is followed by stabilization ponds or settling cells which provide 26.4 days detention time.

The second system is located in Warden, Washington. The design load was 10,900 lbs BOD/day with a flow of 1.1 mgd; three 75 horsepower aerotorswere employed. A 10-day detention time was selected and it is followed by three settling cells which provide a detention time of about 15 days.

Both of these systems are in operation, but operating data are not currently available. It is of interest to note that they both employed a rather long detention time in the aerated lagoon. This was to ensure the conversion of most of the soluble BOD into suspended solids. The long settling times should assure good removal of the suspended solids and thus, good overall treatment.

The third system mentioned earlier has been designed and construction is nearly complete. This work is being partially supported by an FWQA R&D grant to the City of Grand Forks, North Dakota. The waste to be treated consists of domestic plus effluents from three potato processing plants. Flow will range from 3.2 to 4.2 mgd and the BOD load will range from 11,000 to 25,000 lbs/day depending upon number of processing plants in operation. The existing system for the City consisted of four stabilization ponds with a total surface area of 640 acres. An 18-month study will investigate the use of four cells in various combinations to pretreat the waste before discharging into the stabilization ponds. Two of the cells are complete-mix aerated lagoons and two are complete-mix, covered anaerobic units. Each of the four cells in 15 feet deep and has a volume of 8.4 million gallons. The first 12 months of operation call for one-half the flow to go to an anaerobic cell followed in series by an aerated cell. The remaining flow will be split evenly between the other anaerobic and aerobic cells which will be operated in parallel. The last 6 months of the study will involve the use of all four cells in series, two anaerobic followed by the two aerobic units.

Data from full-scale installations will be forthcoming in the next 6 to 12 months at which time the derived constants can be substantiated or revised, if necessary.

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AERATED LAGOON TREATMENT OF LONG ISLAND DUCK WASTES

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Introduction

The production of ducks for slaughter is a large agri-business on Long Island, New York. Approximately seven million ducks are raised in Suffolk County, the easternmost county of Long Island, each year. This production represents about sixty-five per cent of the total restaurant and domestic duck supply of the nation. Other duck production areas are located in Virginia, California, and midwestern states such as Wisconsin and Illinois. While the recorded presence of the duck industry on Long Island can be traced back to 1824, duck production reached its present level by World War II. Today the industry on Long Island is worth approximately 30 million dollars.

The number of ducks raised on Long Island has remained relatively constant for the past 25 years, however, the number of farms has decreased. In 1959, 49 farms were engaged in duck raising (1), while today approximately 35 farms remain. The duck production operations are of varying size but all have essentially the same duck raising procedure. In a typical operation, the ducks are grouped along a flowing channel of water (duck runs) by age group, from hatchery to slaughter. The life of the bird is from six to seven weeks. The water aids in the development of marketable feathers which represents a profitable byproduct from duck production operations.

The shore areas of Suffolk County are the principal location of the duck farms. Sandy shorelines, abundance of fresh water, and a mild climate are the primary reasons for location in this area. Duck farms are generally situated along the streams, coves, and estuaries that characterize this region.

The water in the duck runs carries the wastes of the birds to the treatment systems. A large part of the duck's time is spent out of water and sizable quantities of waste accumulate on the shore and are flushed to the treatment systems by rainstorms and subsequent runoff. Treatment systems therefore must be expected to treat variable flow rates and waste quantities.

Duck farms have traditionally used large quantities of water which has been readily available from ground water and surface supplies. Water usage is variable among the duck farms and has been reported to range from 4 to 96 gal/duck/day (1). Current water usage is near the lower end of this range as duck producers have sought to minimize their waste flows. In the past duck farmers fenced off portions of the open waters to provide areas in which the ducks could swim. Today, as part of pollution control systems many farmers dike or divert surface or ground water from the open waters to supply swimming areas for the birds and to facilitate a controlled wastewater treatment and disposal system. Other farmers have turned to "dry farming" techniques that minimize water use in order to avoid the cost of wastewater treatment.

The uncontrolled discharge of duck wastewater to the waters surrounding the shore of Long Island has caused water pollution problems. Commercial shell and fin fishing have long been important components of the economy of eastern Long Island. During the late 1940's and early 1950's the production of shellfish in the off-shore waters declined considerably. The discharge of untreated duck wastewater was thought to be a major factor in the decline. Wastewater discharged from the duck farms has been documented as having caused high coliform densities, BOD, and phosphorus concentrations in the off-shore waters (2,3). In 1965, closure of 42 per cent of the bay areas due to bacterial contamination was reported to have resulted in a 2.5 million dollar loss for the shellfish industry (4).

Suffolk County has experienced a four-fold increase in population since 1950 and has had the third largest population growth in the country after Texas and California. Past water pollution problems, the increasing demand for domestic and commercial water, and the increasing use of water for recreational activities has placed continued pressure on the duck production industry to maintain and enhance the quality of the environment. The duck producers have responded by working with local and state agencies to provide adequate treatment for their wastewater.

Duck Wastewater Characteristics

Wastes enter the wastewater treatment systems from being deposited in the duck runs, on the land adjacent to the runs, in the shelters along side of the runs, and from slaughtering and processing facilities at some farms. These conditions, plus highly variable flowrates, make concentrations of BOD, suspended solids, phosphates, and nitrogen vary over a wide range. Large amounts of sand, manure particles, and undigested feed characterize duck farm wastewaters. Concentrations, though highly variable, generally are not unsimilar to high strength municipal wastewater concentrations. Characteristics of duck wastes have been reported on a lbs/day/1000 duck on water basis and average values are noted in Table 1. Based on Table 1, and assuming per capita human production of 0.17 lbs BOD/day and 1.5 gm P/day, 8.3 and 1.3 ducks are equivalent to one person on a BOD and phosphorus basis respectively.

Duck production on Long Island is a seasonal business (Figure 1). The duck raising season is from about March through November with the peak period being generally April through October. At most of the farms the duck population decreases to zero during the winter months although some farms maintain breeder stock during the winter. The seasonal production means that any treatment system must be capable of handling the variable flows and waste throughout the year.

Water Pollution Control Efforts

The New York State Water Pollution Law of 1949 provided the impetus for a pollution abatement program for the duck farms. In 1951 receiving waters were classified and the following four phase program for the duck growers was instituted.

Phase 1) Removal of all ducks from the open and public waters.
Phase 2) Removal of settleable solids from effluent.
Phase 3) Treatment of the effluent to reduce the bacteria and cloriform counts to acceptable levels.
Phase 4) Removal of phosphates.

Phase 1 was completed by 1953 by diking or diverting natural water courses to separate duck wastewaters from the open bays. Earthen settling basins were employed to remove settleable solids from impounded waters in an attempt to complete Phase 2. The lack of success with the settling basins and a need for improved BOD reductions led to the introduction of aerated lagoons for duck wastewater treatment in 1964. Phase 3, was accomplished by chlorination of the duck farm effluents since laboratory studies had shown this approach to be economically feasible, especially on aerated lagoon effluent (5).

Phase 4, phosphorus removal, is currently under investigation. The possibility of chemical coagulation, land disposal by spray irrigation, and biological treatment were investigated in 1966 (6). Based on these studies, pilot plant studies were carried out in 1967 and 1968. These investigations failed to establish criteria upon which chemical treatment facilities could be designed. More detailed laboratory and pilot plant studies are underway and are to be completed by early 1971.

Table 1. Average production of wastewater constituents (11)
lbs/day/1000 ducks on water

Volume of wastes - gallons per day per 1000 ducks on water	BOD 5-day	Chlorine Demand	Suspended Solids Total	Suspended Solids Volatile	Nitrogen Total	Phosphate Total	Phosphate Soluble
	Pounds per day per 1000 ducks on water						
Average - 18,500	20.6	1.6	96	58	5.7	7.6	3.6
Maximum - 34,200	30.4	2.1	123	70	6.4	10.5	4.6
Minimum - 10,800	16.5	1.2	52	35	4.7	4.7	2.5

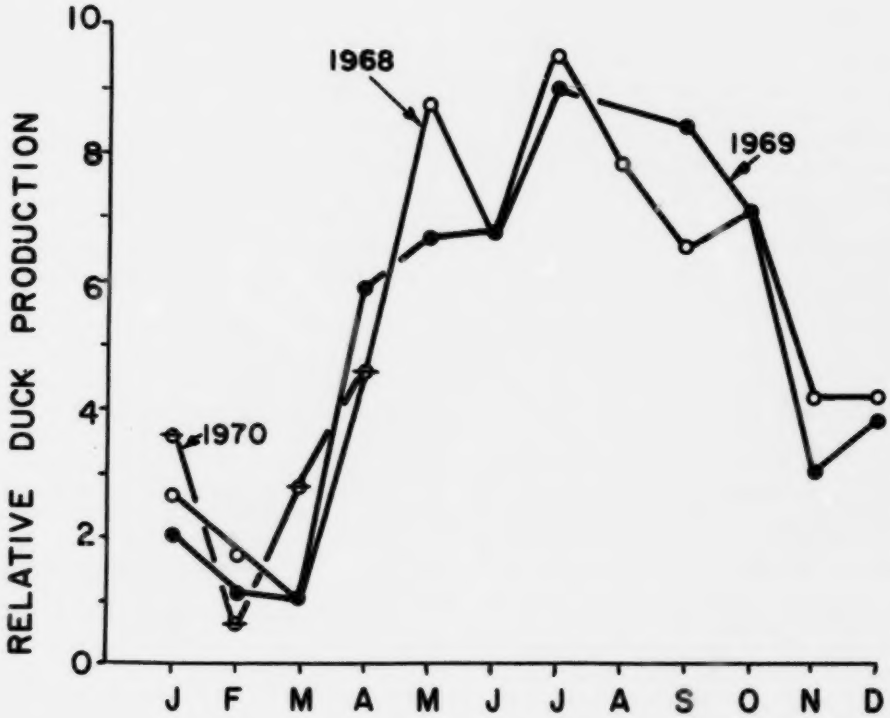

Figure 1. Seasonal Duck production on Long Island

Table 2. Pilot plant study influent characteristics

	90%*	50%*	10%*
BOD_5 (mg/l)	70	130	260
Suspended Solids (mg/l)	140	420	1150
Total Phosphate (mg/l) as P	8.4	18.5	35.5
Soluble Phosphate (mg/l) as P	4.0	11.5	22
Total Nitrogen (mg/l) as N	18	41	68

* Percent of time the noted concentration was exceeded.

To assure adequate control of the duck wastewater, effluent discharge requirements have been specified by the New York State Department of Health in terms of the pounds of BOD and suspended solids that may be discharged. Residual chlorine concentrations in effluents have been specified for set chlorine contact times. Permits to operate waste treatment facilities have been issued to each farm. In addition, weekly analyses of BOD, suspended, and settleable solids are made on aerated lagoon influents and effluents. Dissolved oxygen and the SVI in the lagoons and the pH of lagoon effluents also are measured weekly. Total phosphates in the influent and effluent from the lagoons are measured monthly.

Per cent removal requirements vary from farm to farm due to waste loads and points of discharge. As much as 90 per cent removal of BOD and suspended solids has been required in some cases.

Aerated Lagoon Studies

In 1965 laboratory treatability studies (6) were conducted to develop design criteria for the treatment of duck wastes in a number of possible systems. The results indicated that an aerated lagoon would function satisfactorily with these wastes. Theoretical relationships and the results of the laboratory study indicated that the BOD in the effluent from the aerated lagoon would be due primarily to the suspended solids in the effluent. Accordingly, a solids separation unit, a settling pond, was contemplated following the aerated lagoons. The laboratory results indicated that 90% removal of the influent BOD would be accomplished in an aerated lagoon having five days liquid retention time. The removal could be increased to 95% if a solids separation unit followed the aerated lagoon.

Although the laboratory units were well mixed, it was intended that the aerated lagoons used for the wastes be mixed sufficiently to insure distribution of oxygen and waste throughout the basin but not necessarily to maintain all the solids in suspension. Under these conditions a portion of the solids will settle to the lagoon bottom to undergo stabilization anaerobically.

Prior to constructing aerated lagoon units at all the duck farms, a pilot plant scale treatment study was conducted during the 1967 growing season (February thru November) to confirm the design parameters, treatment efficiency, and reliability of treatment (7). The pilot plant consisted of an aerated lagoon having a capacity of 250,000 gallons, a depth of 8 feet, and was equipped with a 5 horsepower floating aerator. The effluent from the aerated lagoon flowed to one of two earthen settling lagoons, each having a capacity of 25,000 gallons. The settling lagoon effluent flowed to a chlorine contact chamber. The pilot plant was designed to treat 50,000 gallons per day. Difficulties in controlling the flow caused the flow to vary from 19,000 to 90,000 gpd with an average flow of 51,000 gpd.

Composite samples from the pilot plant were taken twice a week and analyzed for a variety of pollutional parameters. Analyses of the influent to the pilot aerated lagoon provided additional information on variability of the influent characteristics during a production season. Probability relationships of BOD_5, suspended solids, total phosphates, soluble phosphates and total kjeldahl nitrogen that were exceeded 10, 50, and 90% of the time are noted in Table 2. As expected, the wastes are nutritionally balanced and require no supplementation of specific nutrients.

The pilot study was successful with an average of over 90% BOD and suspended solids removal being accomplished. Analyses of the effluent from the aerated lagoon and the final effluent from the settling pond corroborated the expectation that the BOD in the effluent was a function of the suspended solids (Figure 3). The flattening of both curves was undoubtedly due to inert solids being washed out rather than biological solids. The settling ponds were alternated when they were to be cleaned. While the ponds apparently functioned satisfactorily, a more positive and continuous solids removal system could have reduced the solids in the final effluent even further.

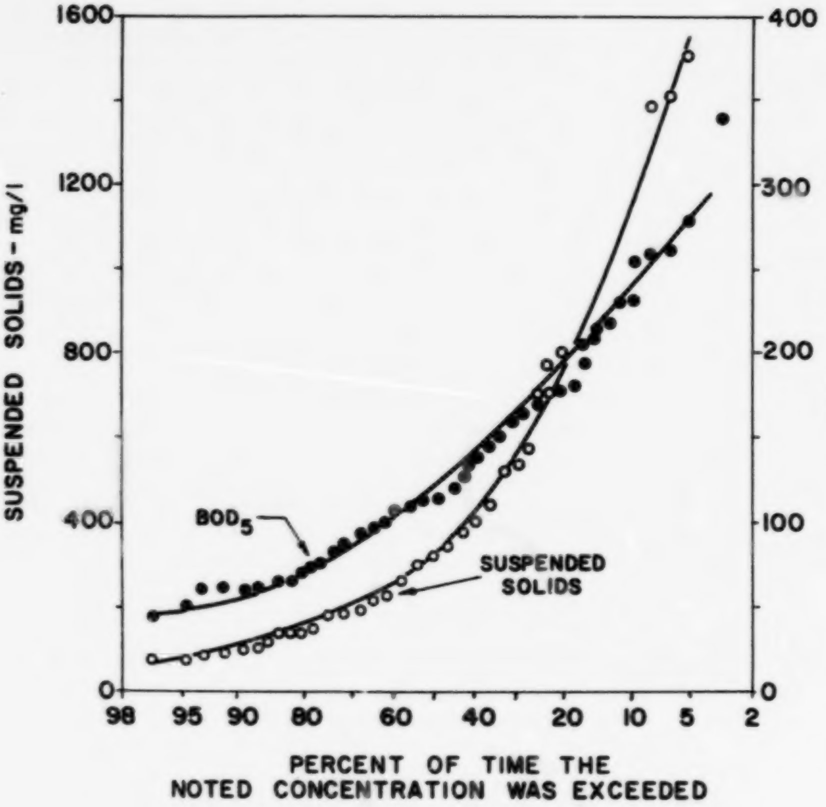

Figure 2. Variability of influent wastewater - pilot plant - 1967

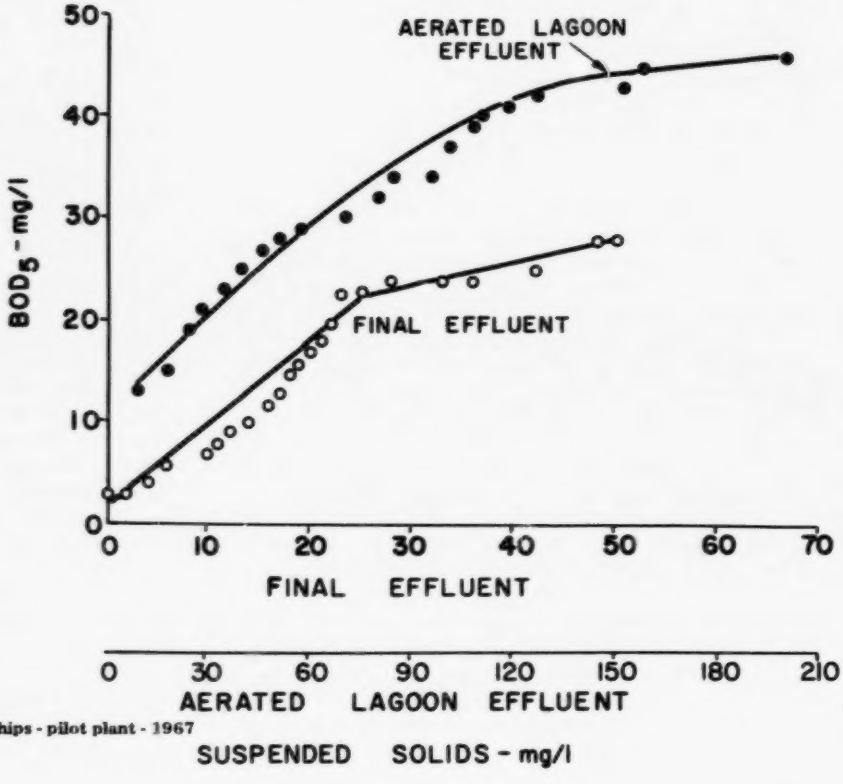

Figure 3. BOD and suspended solid relationships - pilot plant - 1967

Utilization of Aerated Lagoons

Based upon the results of the pilot plant study, each duck farm utilizing a liquid wastewater system constructed adequate aerated lagoon capacity to treat their waste. Most of the farms incorporated settling ponds and all incorporated chlorination of the final effluent. The permit from the New York State Department of Health stipulated the type and size of facilities approved for operation. The approved facilities included pumps, aerated lagoons, settling ponds, aerators, chlorine contact tank, gas chlorinator, and recording flow meter.

A common design for the treatment system was developed. All equipment was obtained in quantity through the Long Island Duck Farmers Cooperative at considerable savings. As noted earlier, the floating surface aerators were sized to maintain distribution of oxygen and waste but not suspension of all the suspended solids. The aerator power relationships varied from 0. 0008 to 0. 04 nameplate HP/1000 gallons lagoon capacity. Suggested power relationships to approach complete mixing concepts with solids in suspension are 0. 027-0. 044 HP/1000 gal (8) while suggested relationships that insure uniform dissolved oxygen but not all suspended solids in suspension are 0. 015-0. 02 HP/1000 gallons (9). Using these relationships, it can be observed that the dissolved oxygen distribution should have been adequate in all lagoons. About 40% of the lagoons had power relationships such that most of the solids should have been kept in suspension.

The Duck Farmers Cooperative also provided funds for the routine analyses required by permits issued by the New York State Department of Health. These analyses are done at the Duck Research Laboratory of Cornell University, at Eastport, Long Island and have been underway since the summer of 1969. BOD analyses are made on grab samples collected one to four times a month. The results of the analyses have been analyzed for two periods, September 1969, during the peak production season and one of the warmer months, and April 1970, the beginning of the growing season and one of the cooler months.

The data shown in Figures 4 and 5 represent data from all 25 farms having aerated lagoons being analyzed in the routine sampling program. The spread of data of the BOD of the lagoon influent indicates the variability in the wastewater characteristics that exists among the duck farms, primarily due to management techniques. The effectiveness of the aerated lagoons and settling ponds can be observed in the difference between the influent and the effluent characteristics (Figure 4) and the per cent BOD removal (Figure 5). In September 1969, 36% of the wastewater aerated lagoon systems on Long Island were removing 90% or greater of the influent BOD and 50% of the systems were removing 85% or greater of the BOD.

Although the aerated lagoons were designed to have an average liquid detention time of five days, the quantity of water entering the aerated lagoons has varied since the lagoons were installed. It should not be inferred that all of the lagoons noted in Figures 4 and 5 had five day detention times. Flow data at the lagoons was not readily available and the actual detention times were not determined. Scattered data indicated that the actual detention time at some of the lagoons may have varied from three to eight days. No correlation of detention time with per cent removal was possible. The variation in actual detention time undoubtedly caused some of the spread in data on Figure 5.

As might be expected in the beginning of the duck production season and cooler weather, the performance of the aerated lagoon systems in April 1970 was somewhat less than that in September 1969. In April 25% of the aerated lagoon systems were removing 90% or greater of the influent BOD and 50% of the systems were removing 70% or greater of the entering BOD. It is expected that the systems will be more effective during the summer of 1970 as the systems reach equilibrium consistent with peak duck production and warmer temperatures.

Phosphate Removal

Evidence indicates that the phosphorus contained in the duck wastes contributes to excessive algae growth and consequent failure of segments of the shellfish industry (2,3). The New York State Department of Health has indicated that a substantial degree of phosphorus in the duck wastewater must be removed in the future. The exact degree of removal has not been stipulated as yet.

The techniques for phosphate removal from any wastewater are still in a developmental stage. Chemical treatment is being considered as the most likely process to accomplish this goal with the duck wastewater. However, the most appropriate type or dosage of chemicals, the optimum location for chemical addition, thy quantity of sludge that will be produced, and best methods to handle and dispose of the sludge have not been determined as yet. Based upon experience at municipal facilities it is expected that the cost of phosphate removal may be almost as much as the cost of the previous treatment systems.

With these items unknown, a combination of studies have developed: a) a mathematical modeling study, b) detailed laboratory studies to develop predictive chemical dose relationships based upon wastewater characteristics, and c) pilot plant studies to verify the modeling and laboratory studies.

Since the cost of substantial degrees of phosphorus removal is likely to be high and little data was available to predict the manner in which chemical treatment should be employed, mathematical modeling was utilized to analyse the costs of alternatives. Minimization of the total annual cost of treatment was regarded as the objective while alternative systems were constrained to meet effluent discharge requirements. Dynamic programming, a technique commonly used in operations research, was adapted to identify least cost system designs and sensitivity of solution costs to variables such as water use, duck concentrations, process efficiencies, equipment costs, treatment requirements, type of chemical, chemical costs, location of chemical addition, and sludge disposal costs (10).

Laboratory analyses were made of the effectiveness of three chemicals, alum, lime, and ferric chloride on samples of untreated and settled duck wastewater and on aerated lagoon effluent. Most of the modeling was done for the specific duck production system from which the samples were taken and for the characteristics of the untreated wastewater at the time of sampling. These characteristics were: 2. 1 MGD, 250 mg/l BOD$_5$, 33 mg/l P, and 300 mg/l suspended solids. The alternative physical, chemical, and biological treatment systems that were investigated are

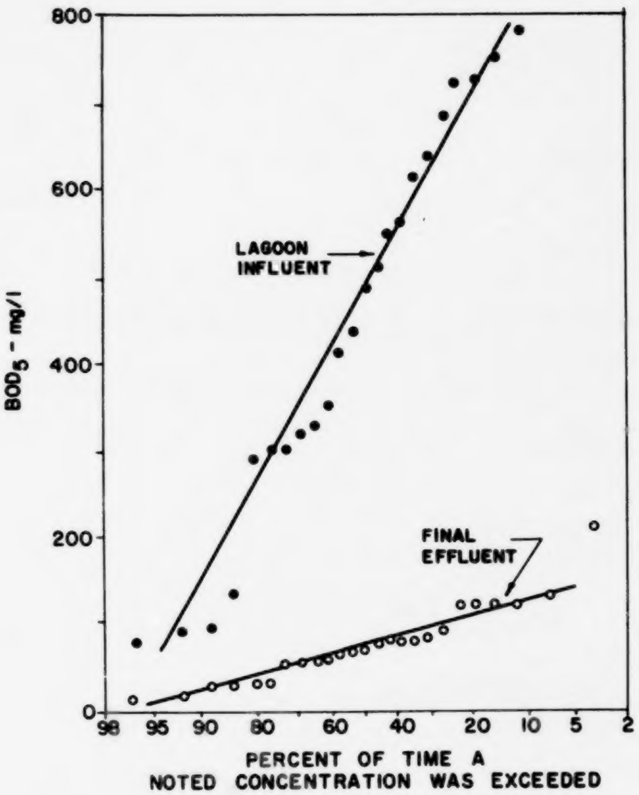

Figure 4. Variability of influent and effluent of aerated lagoon systems for twenty-five duck farms - September 1969

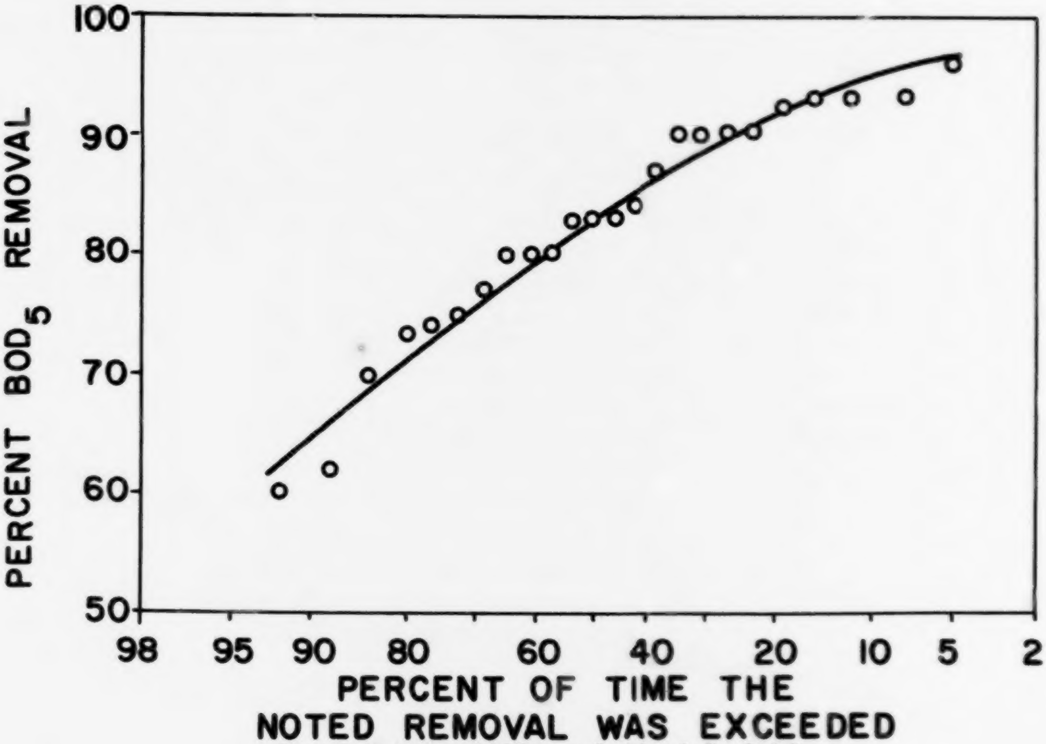

Figure 5. Variability of BOD removals at twenty-five duck farms - September 1969

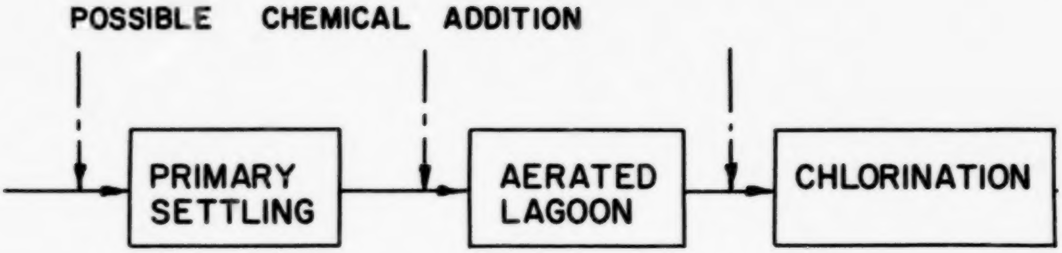
POSSIBLE CHEMICAL ADDITION
PRIMARY SETTLING
AERATED LAGOON
CHLORINATION

Figure 6. Possible points of chemical addition to remove phosphates

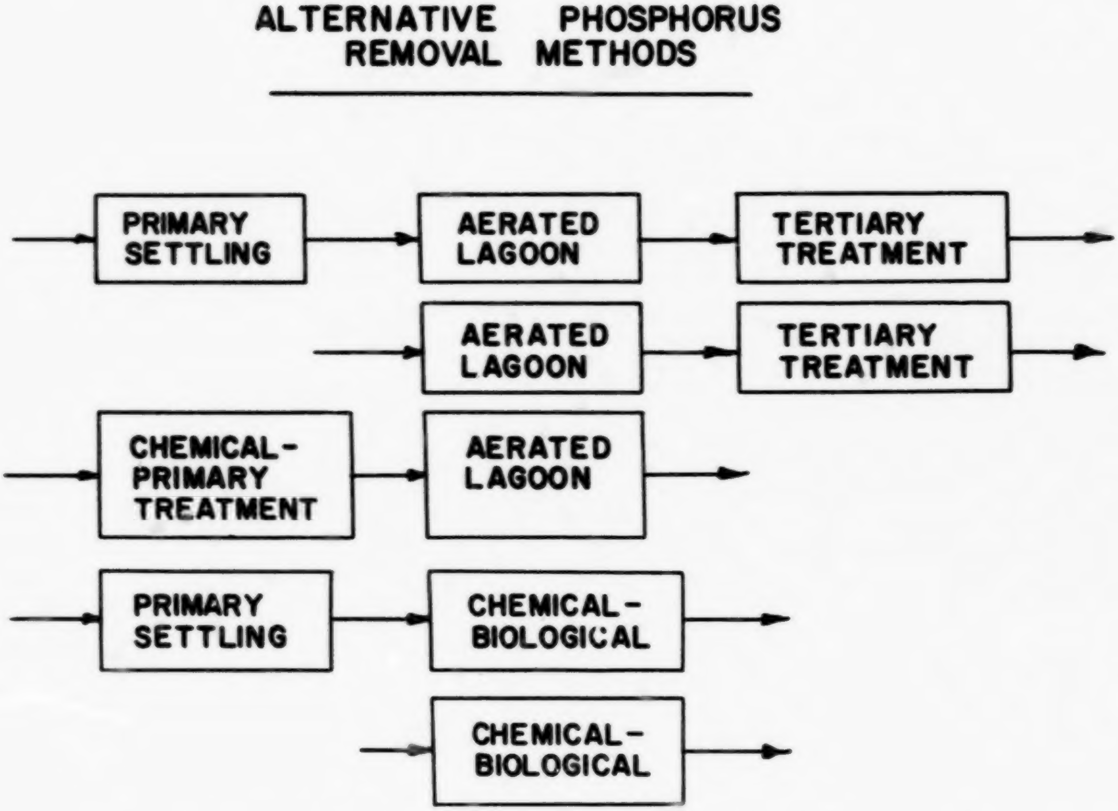
ALTERNATIVE PHOSPHORUS REMOVAL METHODS
PRIMARY SETTLING
AERATED LAGOON
TERTIARY TREATMENT
AERATED LAGOON
TERTIARY TREATMENT
CHEMICAL-PRIMARY TREATMENT
AERATED LAGOON
PRIMARY SETTLING
CHEMICAL-BIOLOGICAL
CHEMICAL-BIOLOGICAL

Figure 7. Alternative phosphorus removal systems for duck wastewaters

noted in Figures 6 and 7. The minimum treatment requirements that the treatment systems were to meet were 80% PO_4, 92% BOD, and 95% suspended solids removals.

The phosphate removal systems, ie. addition of chemicals: a) to raw wastewater followed by a sedimentation unit and an aerated lagoon or by an aerated lagoon alone, b) to settled wastewater being introduced directly to an aerated lagoon, or c) to aerated lagoon effluent (Figures 6 and 7) provided interesting alternatives. Introduction of chemicals to achieve phosphorus removal affects the design of remaining treatment processes such as the aerated lagoon and the settling units. For instance, the bioflocculation in the aerated lagoon may be enhanced by chemical coagulation while improved settleability of microbial solids and adsorption of organic material on precipitant chemical floc will improve overall BOD reductions. If it can be successfully accomplished, integrated chemical and biological processes offer the possibility of both organic (BOD and suspended solids) and inorganic (phosphate) removal without serious modifications to the existing aerated lagoon systems.

Precipitation of phosphates prior to or in the aerated lagoon was not expected to cause nutrient deficiency problems. The average BOD to soluble P ratio in the untreated waste was 11:1 (Table 2) while the soluble BOD fraction was about 50%. Even if the phosphate concentration were reduced to about 1 mg/l as P in a chemical-primary system or in the beginning of a chemical-biological system, the BOD to soluble P ratio would be about 65:1, which is below the 100:1 level generally considered to be the minimum necessary for biological growth. Thus it was expected that aerated lagoon would not be nutritionally deficient in a treatment system incorporating chemical-primary or chemical-biological steps.

The modeling results were based on limited samples and on bench scale tests of duck wastewaters and an example of these results is presented here only to demonstrate the type of information that can be provided by this approach. For instance, over 50 per cent of the total annual cost of the most economical system (primary settling+aerated lagoon+tertiary treatment) was due to the cost of chemicals and allied sludge disposal. The least cost system cost at least 15 to 35 per cent less than alternative treatment systems.

Individuals interested in information in the mathematical model, the variables studied, assumptions made, and results that were obtained may wish to consult reference 10 since the details of these items are beyond the scope of this paper. Mathematical modeling of these systems has provided a common structure with which to evaluate the many factors that affect the economics of wastewater treatment. The data currently being gathered on the effectiveness of chemical, biological, and physical treatment processes for duck wastewaters will be used in further application of the model thus far developed.

Summary

The duck production operations on Long Island have contributed to the pollution problems in the off-shore waters. The local desires for pollution control, the increasing recreational use of the waters, and the influx of urban population have required that the duck farmers develop adequate wastewater treatment systems.

These systems, which now include aerated lagoons, settling ponds, chlorination, and will implement some mechanism of phosphate removal, constitute one of the most sophisticated systems now being utilized by an animal production operation in the United States.

Data from laboratory and pilot plant studies have shown that aerated lagoons of five day detention time can treat the wastes to meet the requirements of the New York State Department of Health. Full scale aerated lagoons designed on the basis of the pilot plant data have performed satisfactorily with 35% of the duck wastewater aerated lagoon systems removing 90% greater of the influent BOD and 50% of the systems removing 85% or greater in September 1969. The aerator power relationships in the actual aerated lagoons varied from 0. 008-0. 04 nameplate horsepower per 1000 gallons lagoon capacity.

Studies are underway to develop an integrated chemical and biological treatment system to remove phosphates as well as organic matter. Mathematic modeling studies were used to estimate the costs of alternative treatment systems. Based upon limited samples and bench scale tests of duck wastewaters, the most economical system appeared to be primary settling plus an aerated lagoon plus tertiary chemical treatment. With this system, over 50% of the cost was due to the cost of the chemicals and allied sludge disposal.

Laboratory and pilot plant studies are being conducted to provide additional information and verify the least cost system before phosphate removal processes are combined with the current aerated lagoon treatment systems.

Acknowledgements

Mr. William Cosulich provided the reports describing the laboratory and pilot plant studies. The Long Island Duck Farmers Cooperative is sponsoring the current laboratory and pilot plant phosphate removal studies. Mr. Kenneth Johanson is in charge of the treatment system routine analysis program and of conducting the current phosphate removal studies. The assistance of all that have been involved is greatly appreciated A FWQA Research Fellowship permitted the development and completion of the mathematical modeling studies.

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A COMPARATIVE STUDY OF AERATED LAGOON TREATMENT OF MUNICIPAL WASTEWATERS

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INTRODUCTION

In 1971 the Metropolitan Corporation of Greater Winnipeg will be constructing additional treatment works. These works will bring the total treatment capability to a point where all sewage generated in the Metro area can be treated. Prior to making a decision on the type of works, studies were made into various forms of treatment. It was found that there may be some economic advantage to aerated lagoons. However, there was little documented information on aerated lagoons operating in Canadian Prairie winter conditions. For these reasons the Corporation constructed on a pilot scale, three aerated lagoons. The work was supported by Canadian Public Health Grant No. 606-7-167.

Metropolitan Winnipeg has a population of 500,000 and is located at latitude 49° 54' N, longitude 97° 15' W. The climate is of the continental type, the coldest month is January with an average temperature of 2°F below, the warmest month is July with an average temperature of 67°F above, and the average frost-free period (32°F) is 115 days. The average winter snowfall is 51 inches.

During the summer and fall of 1967 the lagoons were constructed; they are of the aerobic-anaerobic type. For economy of construction the pilot cells were located in the corner of an existing stabilization pond. The influent to the systems is domestic sewage from a separate system. The three aeration systems installed are:

1) Air-Aqua*
2) Mechanical Surface Aerator**
3) Air-Gun***

Each system has a capacity of 0.5 Imgd, and is treating domestic waste. The general arrangement is shown on Figure 1 and the design data in Table 1.

EQUIPMENT DESCRIPTION

Air-Aqua

This system operates on the diffused air principle. A 30-HP compressor supplies air to polyethylene tubes laid in a tapered grid on the cell bottom. The system has a 30-day retention time, an operating depth of 10 feet and is divided into two cells, operating in series. Ten percent of the effluent is returned to the inlet for seed.

Surface Aerator

This system consists of eight 20-HP aerators installed in series along the lagoon basin. The basin is sized for a 20-day retention time and has an operating depth of 11 feet. The raw sewage is fed in adjacent to the first aerator only.

Air-Gun

A combination of the diffused air and surface aerator principles is used for this system. There are 54 guns installed in a tapered grid pattern. A 40-HP compressor delivers air into an inverted siphon in the gun base, where a large bubble is formed which rises inside the gun, pushing water ahead and exploding at the surface. The system has a 20-day retention time and an operating depth of 17 feet.

RESULTS

Effluent Quality

The raw sewage concentration for the period January 1st, 1968 to September 30th, 1969 averaged 175 mg/l and 188 mg/l for BOD$_5$ and SS respectively. Corresponding

*As manufactured by Hinde Manufacturing Limited, Hamilton, Ontario.

**Equipped with Lightnin Aerators as manufactured by Greey Mixing Equipment Limited, Toronto, Ontario.

***As manufactured by Aero Hydraulics Corporation, Montreal, P.Q.

Table 1

Summary of Design Date

ITEM	VALUE AND UNITS
Average Design Flow	0.5 Imgd
Influent 5-Day BOD 20°C	250 mg/l
Influent Suspended Solids 20°C	180 mg/l
Deaeration Coefficient (Base 10) k_1 at 0°C	0.13 per day
Deaeration Coefficient (Base 10) k_1 at 20°C	0.50 per day
Temperature Coefficient (applied to k_1)	1.072 per °C
Oxygen Utilization factor a' (lbs. oxygen required per lb. 5-day BOD removed)	1.50
Oxygen transfer ratio (ratio of O_2 transfer to waste to that of water) (α factor)	0.85
Saturation value of waste compared to H_2O (β factor)	0.95
Solubility of oxygen 20°C (780')	9.02 mg/l
Operating dissolved oxygen	2.00 mg/l
Effluent temperature - winter	32°F
- summer	75°F
Influent temperature - winter	48°F
- summer	65°F
Mean ambient air temperature - winter	-16°F
- summer	+75°F
Treatment efficiency required	90%
Retention time - Air-Aqua	30 days
- Surface Aerator	20 days
- Air-Gun	20 days
Operating depth - Air-Aqua	10 ft.
- Surface Aerator	11 ft.
- Air-Gun	17 ft.
Volume - Air-Aqua	15 x 10^6 gal.
- Surface Aerator	10 x 10^6 gal.
- Air-Gun	10 x 10^6 gal.
Mixing requirements for surface aerators	0.016 HP/1000 gals.
Process loading - a) Design	
- Air-Aqua	0.52 lbs. BOD_5/1000 ft^3/day
- Surface Aerator	0.78 lbs. BOD_5/1000 ft^3/day
- Air-Gun	0.78 lbs. BOD_5/1000 ft^3/day
- b) Actual	
- Air-Aqua	0.37 lbs. BOD_5/1000 ft^3/day
- Surface Aerator	0.55 lbs. BOD_5/1000 ft^3/day
- Air-Gun	0.55 lbs. BOD_5/1000 ft^3/day

effluent quality for the same 21 month period was:

	BOD_5 mg/1	S.S. mg/1
Air-Aqua	37	34
Surface Aerator	38	39
Air-Gun	34	34

Figures 2 and 3 show these in detail.

The effect of cold weather on BOD_5 removal is evident when the winter of 1968-69 is compared to the summer of 1968 on the figure 2. The loss of efficiency during the summer of 1969 is attributed to sludge which will be discussed later.

The results were analyzed statistically for the 12 month period September 30th, 1968 to September 30th, 1969. (See Fig. 4 & 5). This period was selected as the most typical for continuous operation.

The median value of the effluent BOD for the three systems ranged between 39 and 38 mg/1. On 10% of the occasions the effluent $>$ 78 mg/1.

Suspended Solids removal remained reasonably constant over the 21 month period, with the exception of start-up, which can be attributed to initial erosion and suspension of materials from construction. On the 12 month basis the median SS in the effluent ranged between 21 and 23 mg/1 and on 10% of the occasions $>$ 48 mg/1.

Figures 10, 11 and 12 in Appendix A give results on temperature, grease and D.O.

Nutrients

Figures 6 and 7 show the average performance of the aeration systems in the treatment of algal nutrients, phosphorus and nitrogen. The average percent removals over the test period were:

System	Nutrient Removal	
	Total Nitrogen (N)	Phosphate (PO4)
Air-Aqua	12.0%	16.9%
Surface Aerator	14.6%	19.5%
Air-Gun	10.0%	23.3%

Although there was a reduction of total phosphorus through all systems the orthophosphate concentrations in the effluent increased. The nutrient removal efficiencies are relatively low compared to results on ponds reported by others. (1) The conventional activated sludge plant operated by the Corporation has removal rates of 36% and 45% for nitrogen and phosphate respectively. There was no appreciable difference in removal rates between summer and winter.

The aeration systems are basically equal in nutrient removal and relatively ineffective.

Temperature

The effluent temperature follows the ambient temperature curve closely as it is almost independent of the raw sewage temperature (See Fig. 10, Appendix A). There is a four month period during the year when the effluent temperature is between 0°C and 1°C.

One of the concerns in design was the possible freezing of the Air Gun cell as little information was available on heat loss through ice cover. To keep the heat loss at a minimum the surface area was reduced by making the side slopes steeper. Observations proved however that the cell had a built-in self protection system. When the temperature rose to 10°F the ice melted on 25% of the cell, but as soon as the temperature dropped the cell would cover with ice to conserve heat.

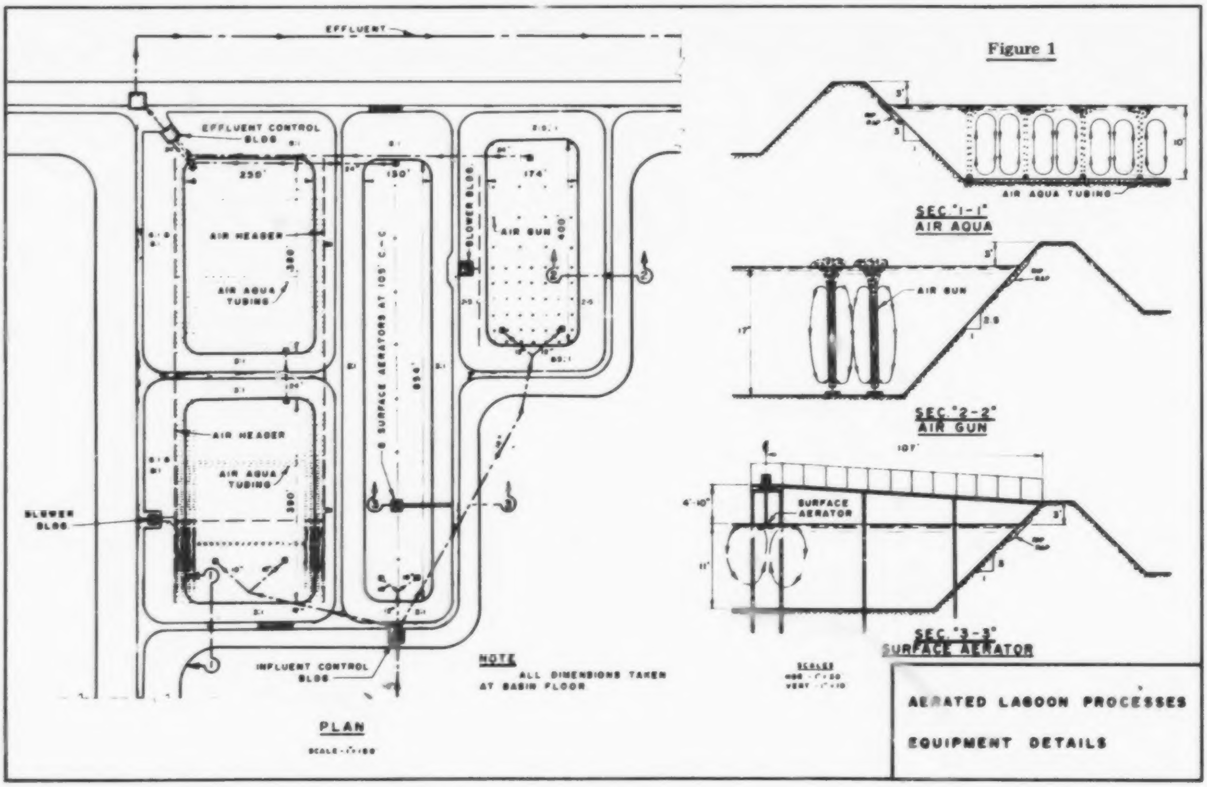
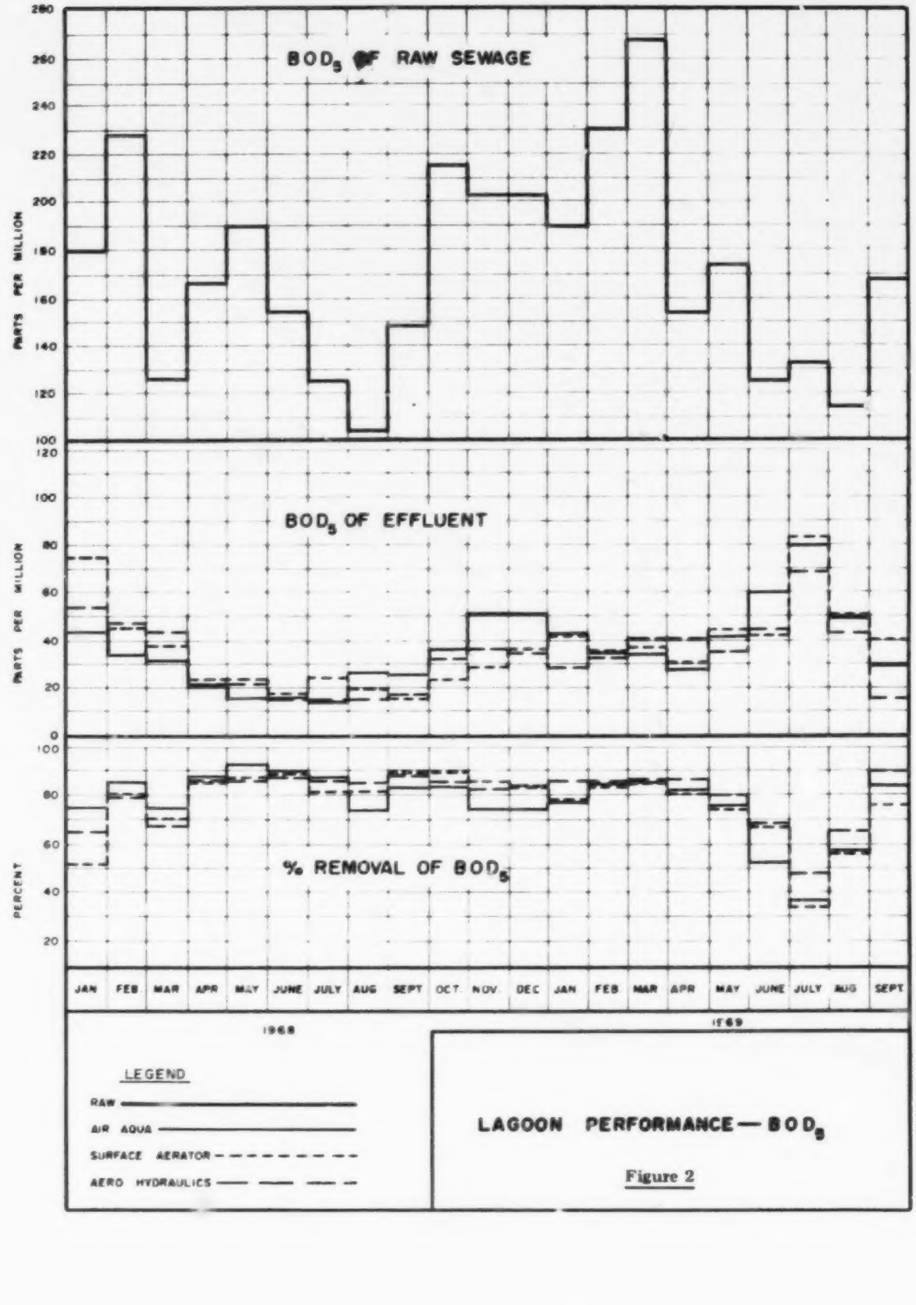
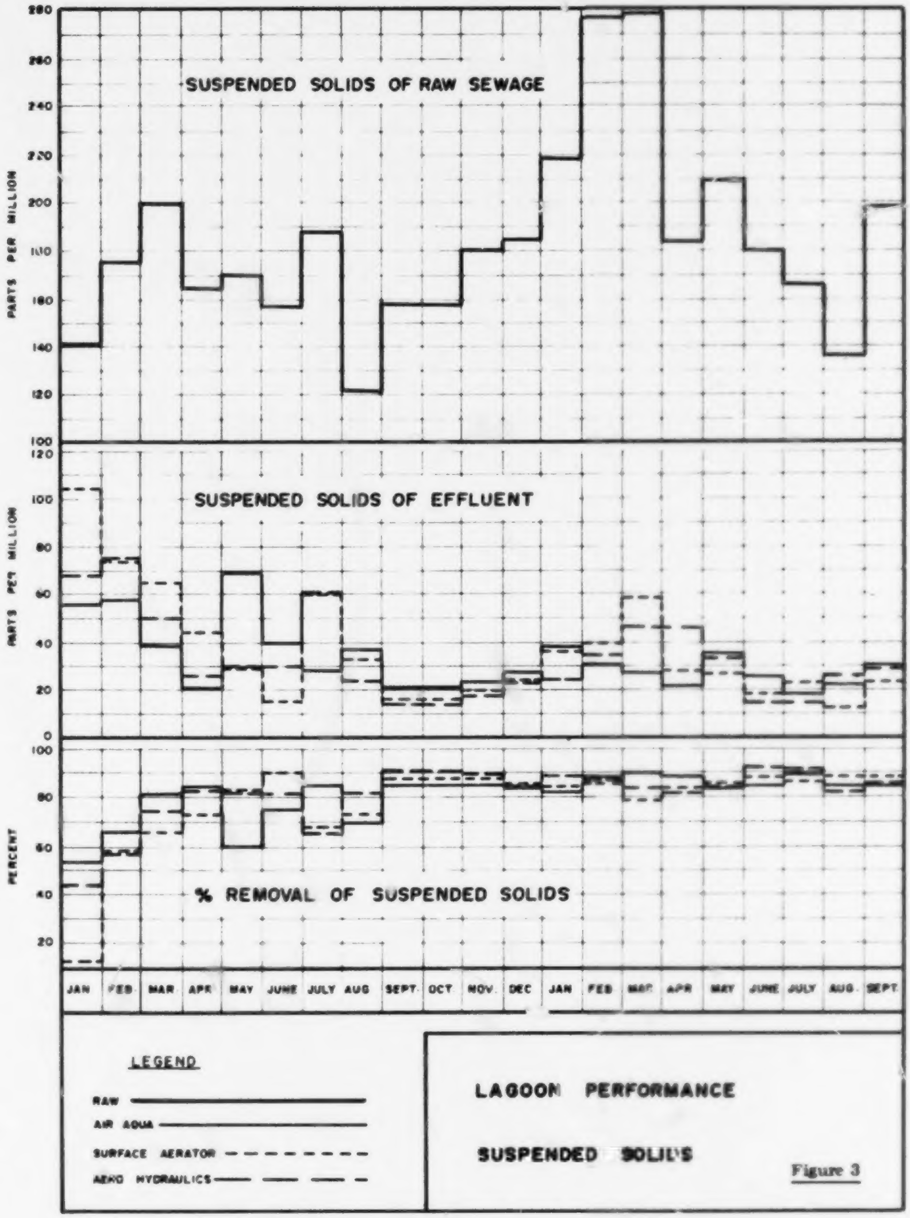

EFFLUENT
EFFLUENT CONTROL BLDG
250'
150'
174'
AIR HEADER
AIR AQUA TUBING
380'
8 SURFACE AERATORS AT 105' C-C
856'
BLOWER BLDG
AIR GUN
400'
BLOWER BLDG
INFLUENT CONTROL BLDG
NOTE
ALL DIMENSIONS TAKEN AT BASIN FLOOR
PLAN
SCALE
3'
10'
AIR AQUA TUBING
SEC. "1-1"
AIR AQUA
3'
AIR GUN
17'
2.5
SEC. "2-2"
AIR GUN
107'
SURFACE AERATOR
4'-10"
3'
11'
SEC. "3-3"
SURFACE AERATOR
SCALES
AERATED LAGOON PROCESSES
EQUIPMENT DETAILS

Figure 1



LAGOON PERFORMANCE — BOD$_5$

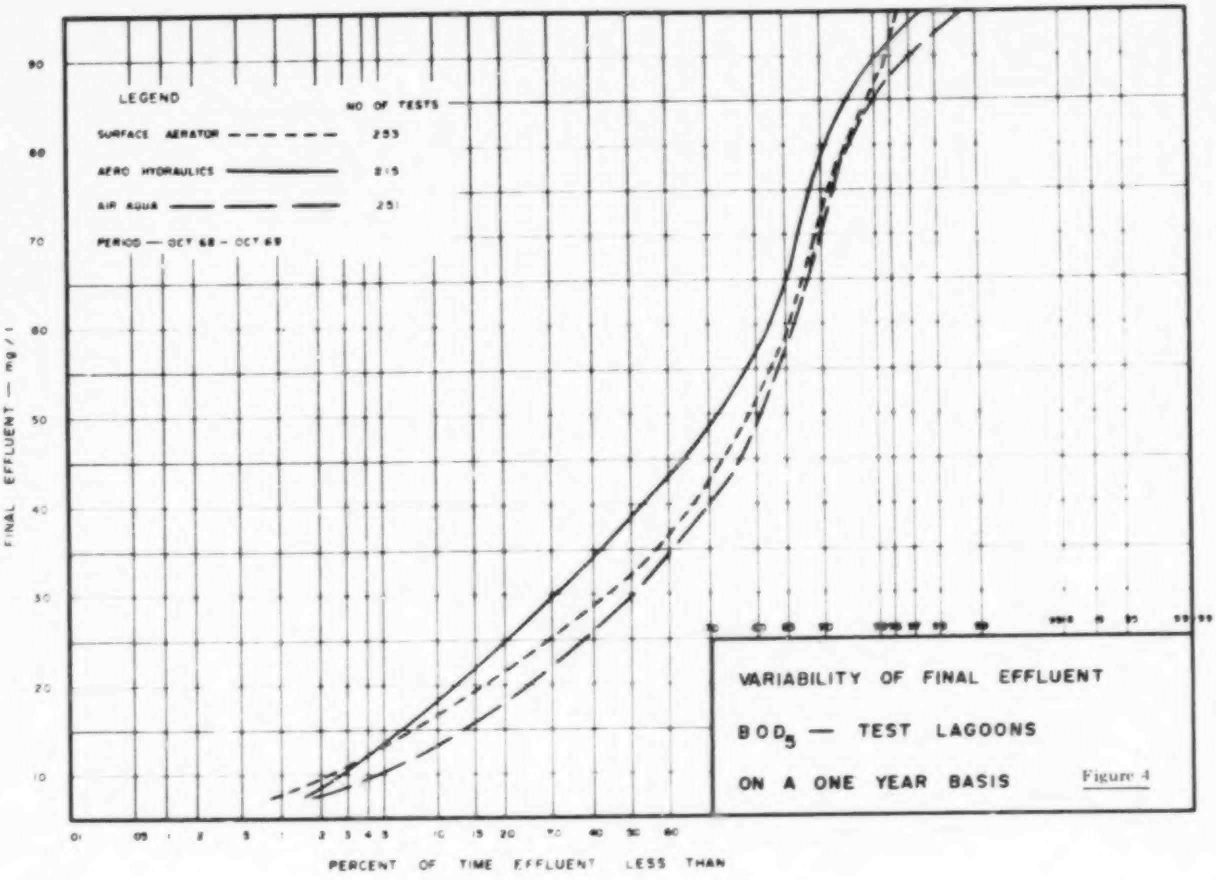
Figure 2



LAGOON PERFORMANCE

SUSPENDED SOLIDS

Figure 3

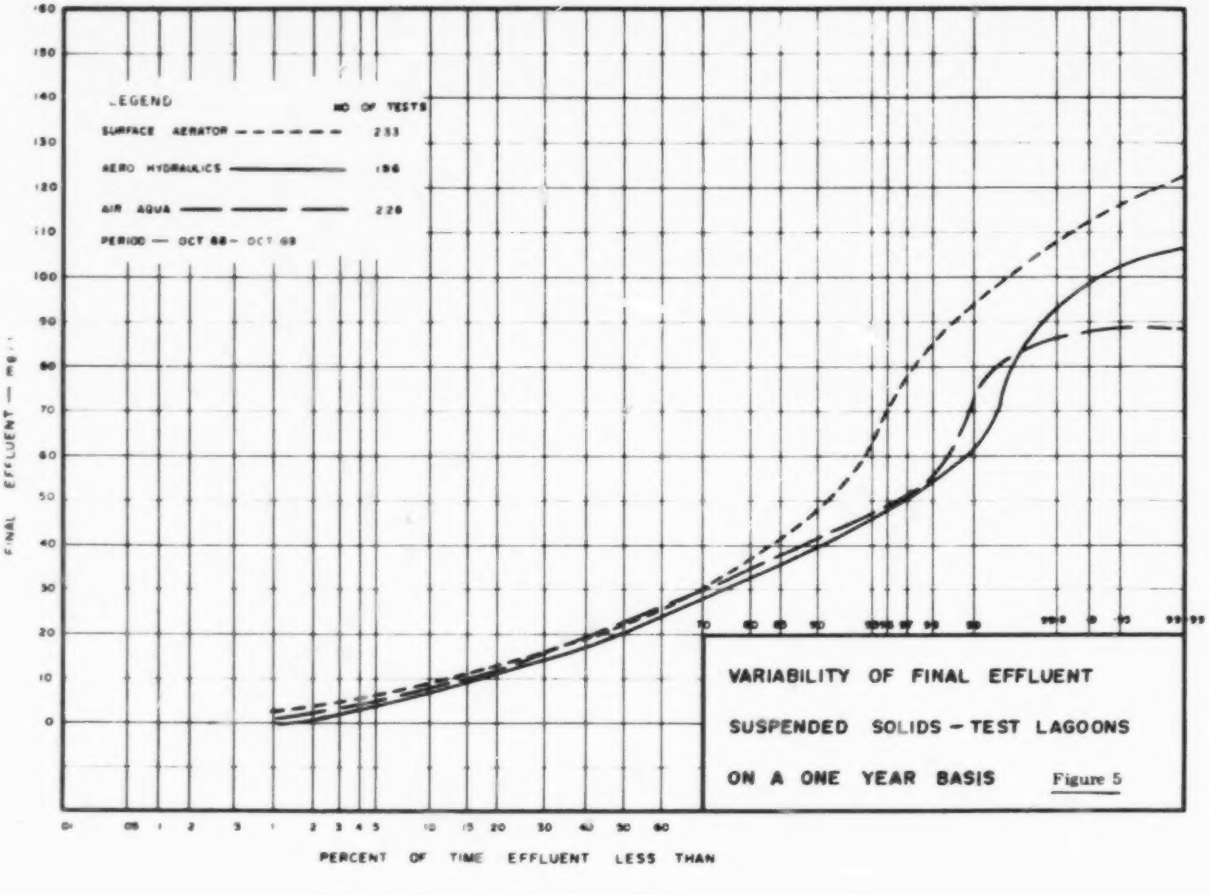


VARIABILITY OF FINAL EFFLUENT

BOD_5 — TEST LAGOONS

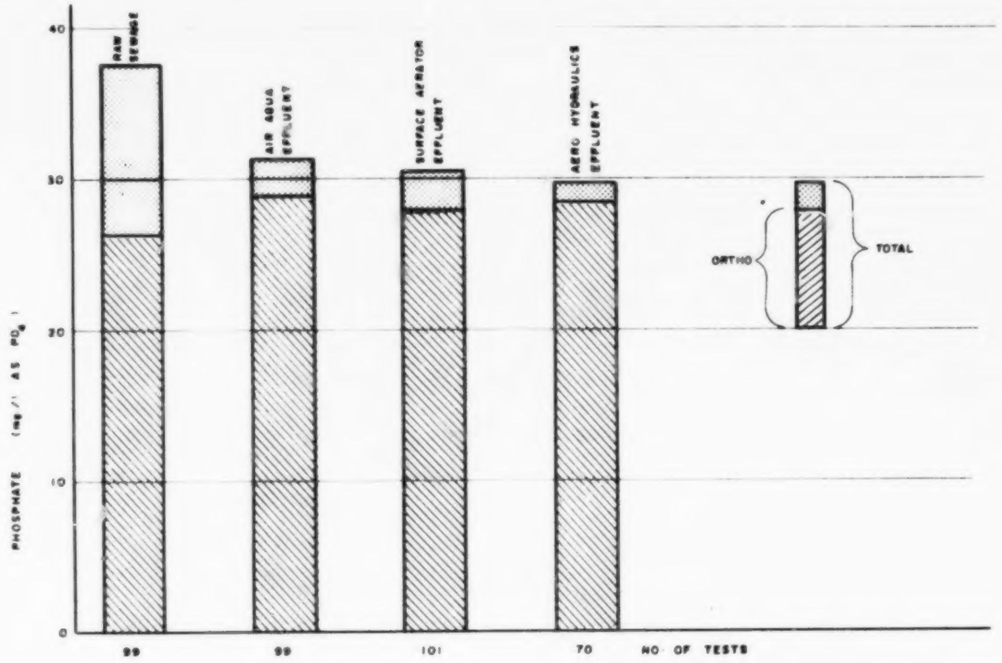
ON A ONE YEAR BASIS

Figure 4



VARIABILITY OF FINAL EFFLUENT SUSPENDED SOLIDS – TEST LAGOONS ON A ONE YEAR BASIS

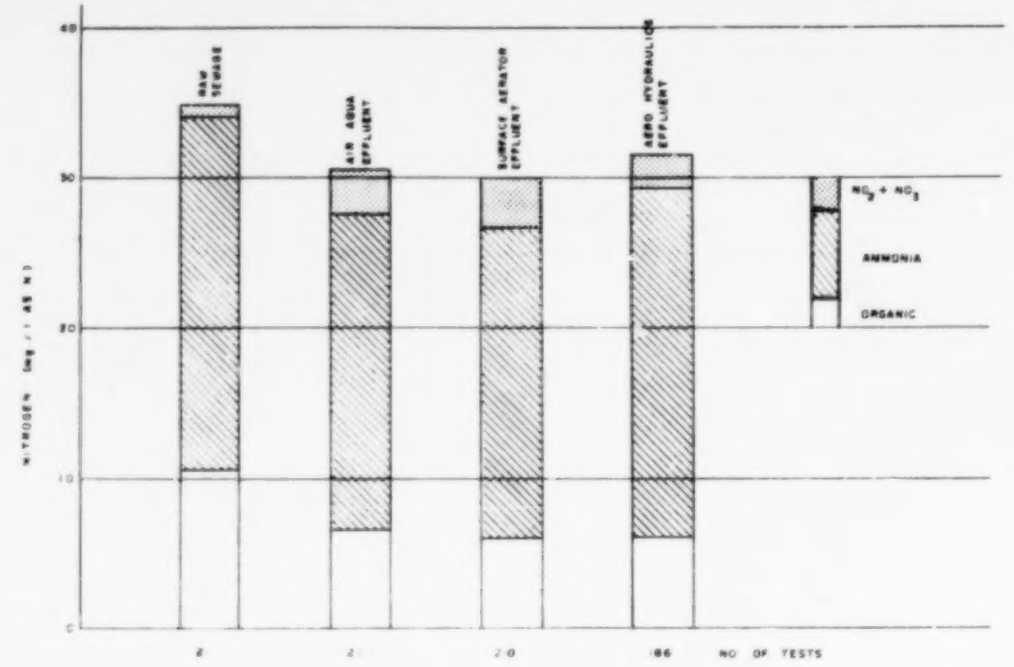
Figure 5



TEST AERATED LAGOONS

AVERAGE TEST RESULTS

PHOSPHATES Figure 6



TEST AERATED LAGOONS

AVERAGE TEST RESULTS

NITROGEN Figure 7

SLUDGE ACCUMULATION

The three aeration systems have shown a substantial build-up of bottom sludges. Samplings were conducted during July 1968, in the fall of 1969 and early in 1970. The accumulation of sludge in aerated lagoons has been recognized by others (2)(3)(4); however the significance of the accumulation of sludge deposits under climatic conditions similar to those experienced in the Canadian prairies has not been reported.

The rate of sludge accumulation in the Air Aqua and Aero-Hydraulics has been estimated to be approximately 0.18 to 0.25 lbs. of dry solids per capita per day (based on population equivalents). The rate is comparable to the 0.21 lbs. of dry solids per capita per day pumped to digestion at an activated sludge plant operated by the Corporation.

Theoretical Sludge Accumulation

Theoretically the overall digestion rate is sufficient to reduce the amount of volatile sludge accumulated to a relatively stable content each summer. After the first summer of operation an annual cycle should occur with the sludge accumulation reaching a maximum in March-April, decreasing to a minimum level in August-September. The theoretical cycle is shown in Figure 8 (based on temperature corrected anaerobic digestion rates).

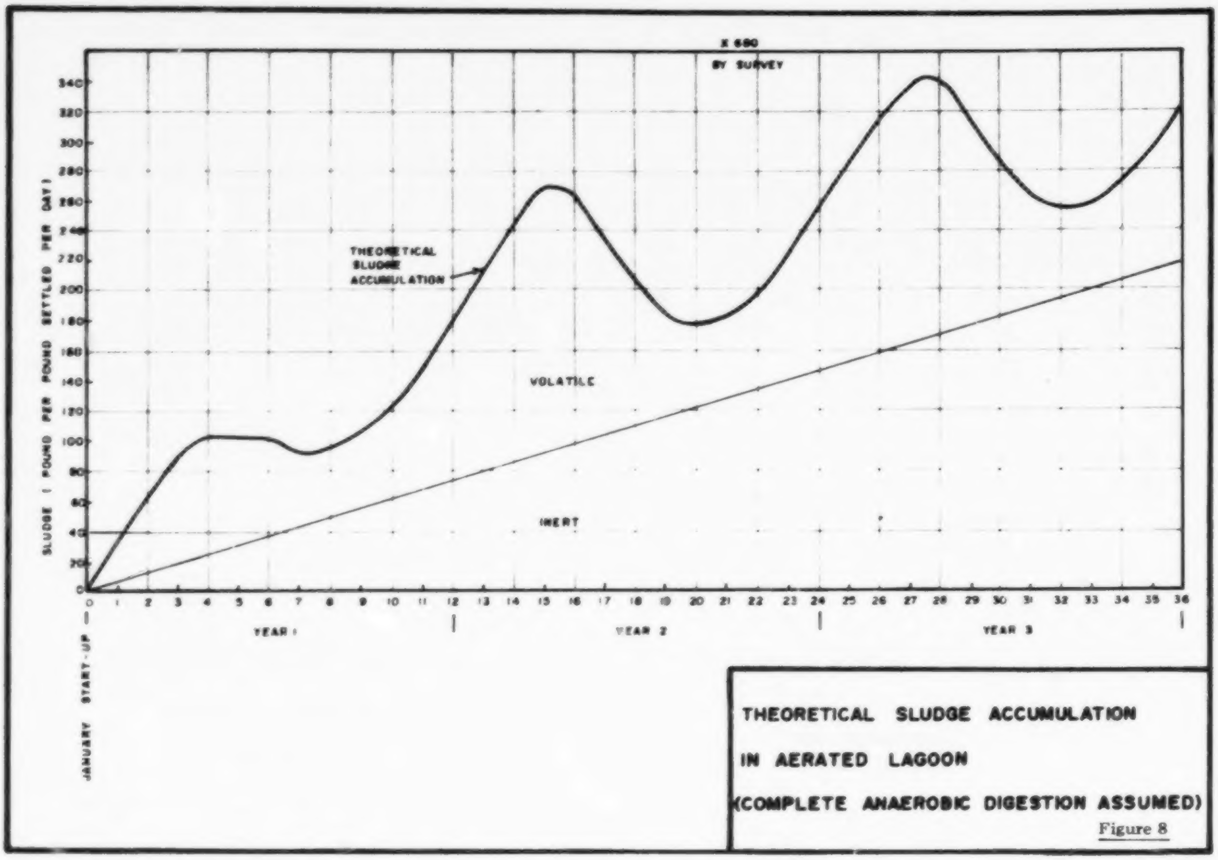
Based on the accumulation of all suspended solids removed, and 0.45 pounds of solids synthesized per pound of BOD_5 removed, the daily sludge production would be 1120 pounds. If the end products of anaerobic digestion had been removed from the system, the amount of sludge accumulation at the September 1969 sampling would have been an estimated 200,000 pounds (average daily accumulation of approximately 300 pounds).

Observed Sludge Accumulation

Based on actual surveys the sludge accumulation (as dry solids) to September 5th, 1969 is estimated as follows:

	Total Lbs.	Lbs/day	Lbs/c/day
Air-Aqua Primary	740,000	1120	0.21
Air Aqua Secondary	145,000	220	0.04
Air-Gun	580,000	930	0.18

Sampling of the Air Gun had to be done through 15 feet of supernatant liquor and these figures are subject to confirmation. Only two samples were obtained in the Air Aqua secondary so the reliability of the 145,000 pound estimate is poor. Considering the methods of obtaining the data it was concluded that there was no apparent difference in the quantities of sludge between the systems, the significant point being that the accumulation is considerably greater than expected. An analysis of the sludge from the Air-Aqua system indicated a moisture content of 90% for the primary cell and 92% for the



THEORETICAL SLUDGE ACCUMULATION
IN AERATED LAGOON
(COMPLETE ANAEROBIC DIGESTION ASSUMED)

Figure 8

secondary cell, the volatile content was 55% for the primary and 45% for the secondary.

Effect of Sludge Accumulation on
Aerated Lagoon Performance

During the months of May, June, July and August 1969, in all three systems, there were significant upward trends in the effluent BOD, as shown in Figure 9. All three systems showed similar relationships. This decline in BOD removal efficiency was accompanied by a trend to reduced dissolved oxygen concentrations in all cells. The reduced dissolved oxygen could be an "effect" caused by high oxygen demands imposed by the end-products of anaerobic decomposition of the sludge, or it could be a "cause" of higher effluent BOD due to an oxygen deficiency.

Some of the related factors which may account for the sludge accumulation and the effect on performance are:

a) release of sludge digestion end products to the mixed liquor,

b) recycle of sludge by bacterial and algal synthesis in the mixed liquor,

c) relatively short period of higher rate anaerobic digestion,

d) insufficient air supply.

The duration of the study has been insufficient to allow definition of the ultimate extent of sludge accumulation and loss of efficiency during the summer. However, based on the observations made it appears that sludge is accumulating at the rate of approximately one ton of dry solids per Imgd treated.

To determine if an abnormal quantity of inert material was contained in the raw sewage or effluent a series of tests was conducted. The volatile content of the raw sewage is normal and shows no evidence of extraneous inert matter. Similarly, the effluent volatile solids are typical for biologically treated sewage.

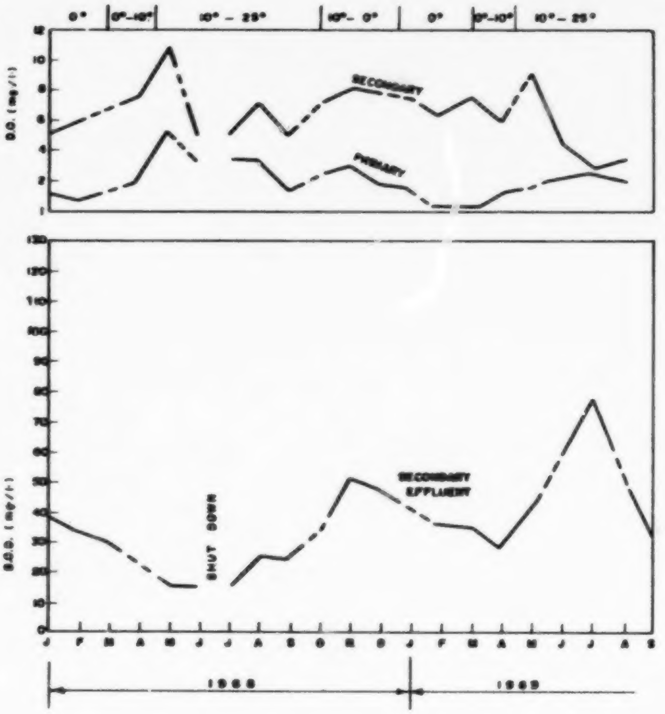
Insofar as the higher BOD in the effluent is concerned, it is our opinion that sufficient additional air can be added economically to handle the benthic demand.

The major problem would appear to be the physical accumulation of the sludge, which will ultimately require removal and additional treatment.

The problems encountered are similar to experience with Imhoff tanks, with respect to removal and storage of BOD and solids during the winter months and the release of soluble BOD and nutrients as acid fermentation of accumulation sludge deposits develop. (5)

OPERATIONAL OBSERVATIONS

Observations were made on mosquitoes, weeds, grease and scum, odours, ice, foaming, grit, erosion and equipment operation. These items are discussed in Appendix B. However, the equipment operation is worthy of discussion now. Plugging of both the Air-Aqua tubing and the Air-Guns occurred. The problem was corrected by re-punching and acid-cleaning the Air-Aqua tubing, and removing and replacing with a new design the siphon chambers of the Air-Guns. At the time of writing both systems appear to be operating satisfactorily, but more time



CHARLESWOOD LAGOONS

MONTHLY TRENDS

AIR AQUA SYSTEM

Figure 9

is required to assess their ultimate reliability. Ice is a problem with the operation of the surface aerator. The basic weakness was the ice build-up on the impeller shaft and supporting surface structure, with subsequent freeze-up and stopping of the mechanism. Although steps were taken to try and eliminate the ice problem it was found that during the winter months only 50% of the surface aerators could be kept operating.

Information on capital and operating costs are provided in Appendix C.

CONCLUSION

Aerated lagoons were found to be capable of providing "secondary equivalent" sewage treatment. There is a problem of sludge accumulation, probably due to prairie climatic conditions. During the summer months a decline in efficiency of BOD removal and a reduction in the dissolved oxygen concentration occurs because of this sludge accumulation.

The economic feasibility of aerated lagoons is questionable until the extent and cost implications of sludge removal are better defined by further research in this area.

The use of surface aerators under prairie winter conditions is not practical due to ice build-up.

It can be concluded that aerated lagoons are an effective means of providing secondary treatment but some provision must be made for sludge handling.

We intend to continue with our work in this area, on a somewhat reduced scale, with an aim to finding an economic answer and determining the long term effects. In addition the suppliers are actively pursuing a solution.

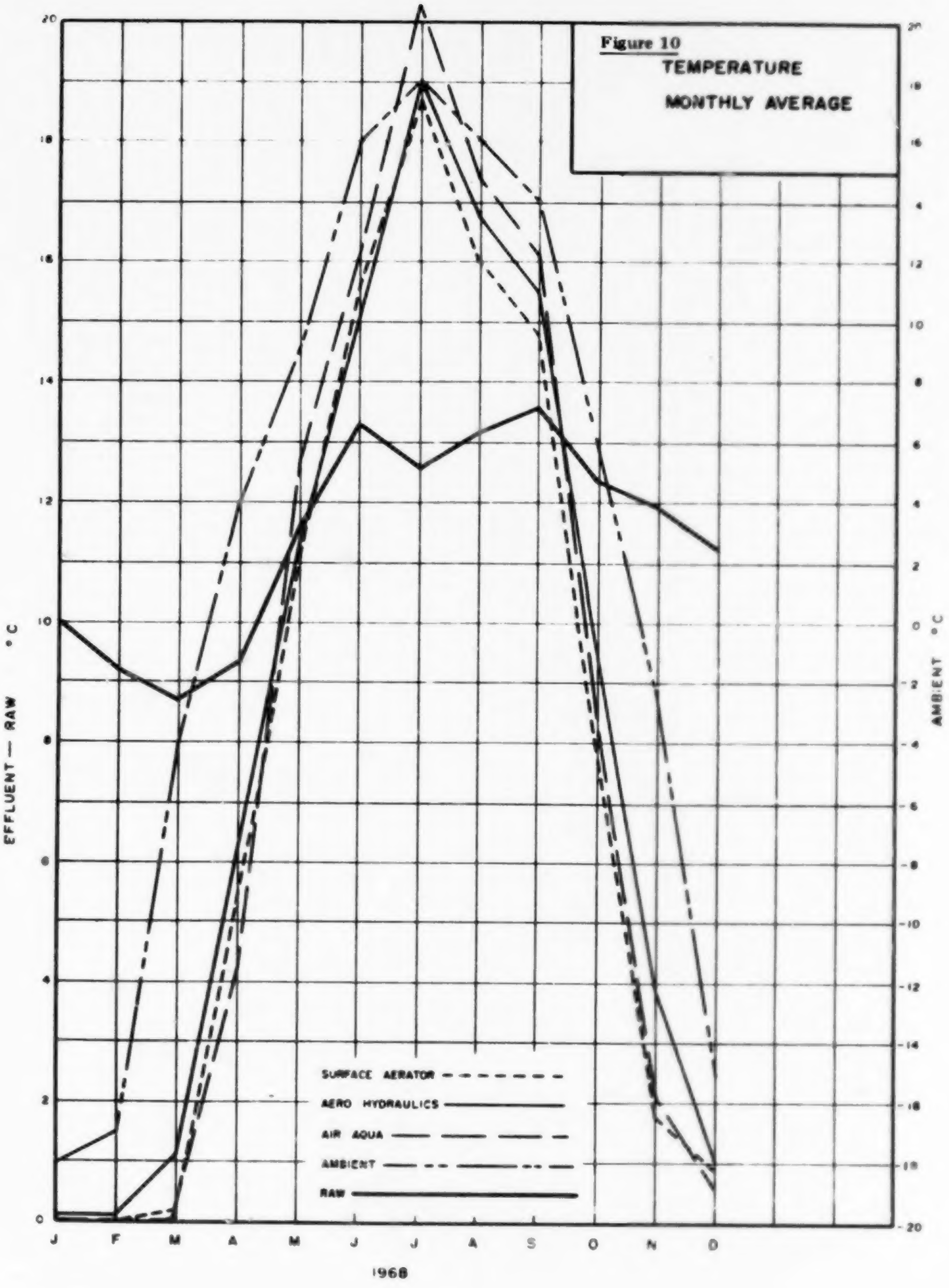


Figure 10
TEMPERATURE
MONTHLY AVERAGE

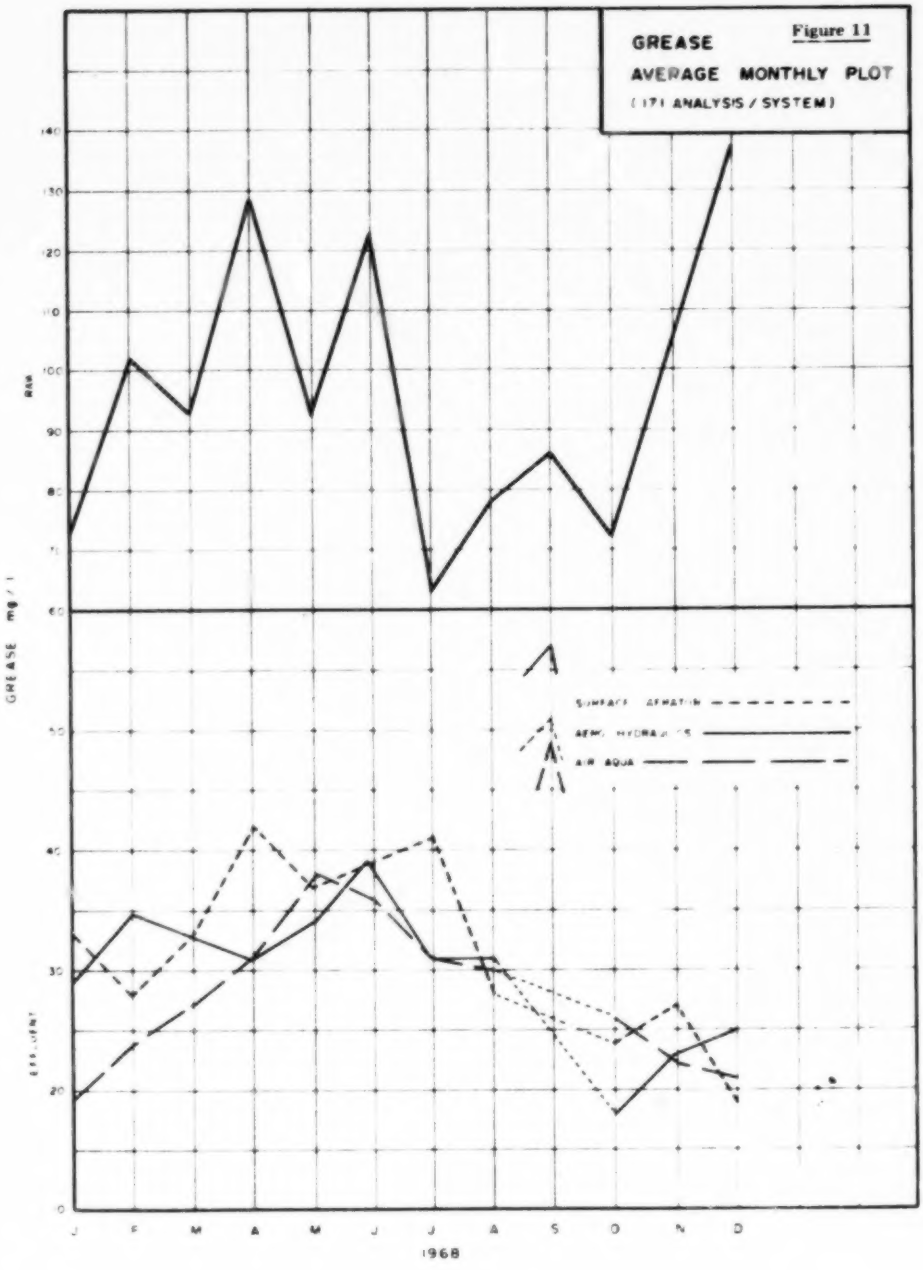


Figure 11

GREASE

AVERAGE MONTHLY PLOT

(171 ANALYSIS / SYSTEM)

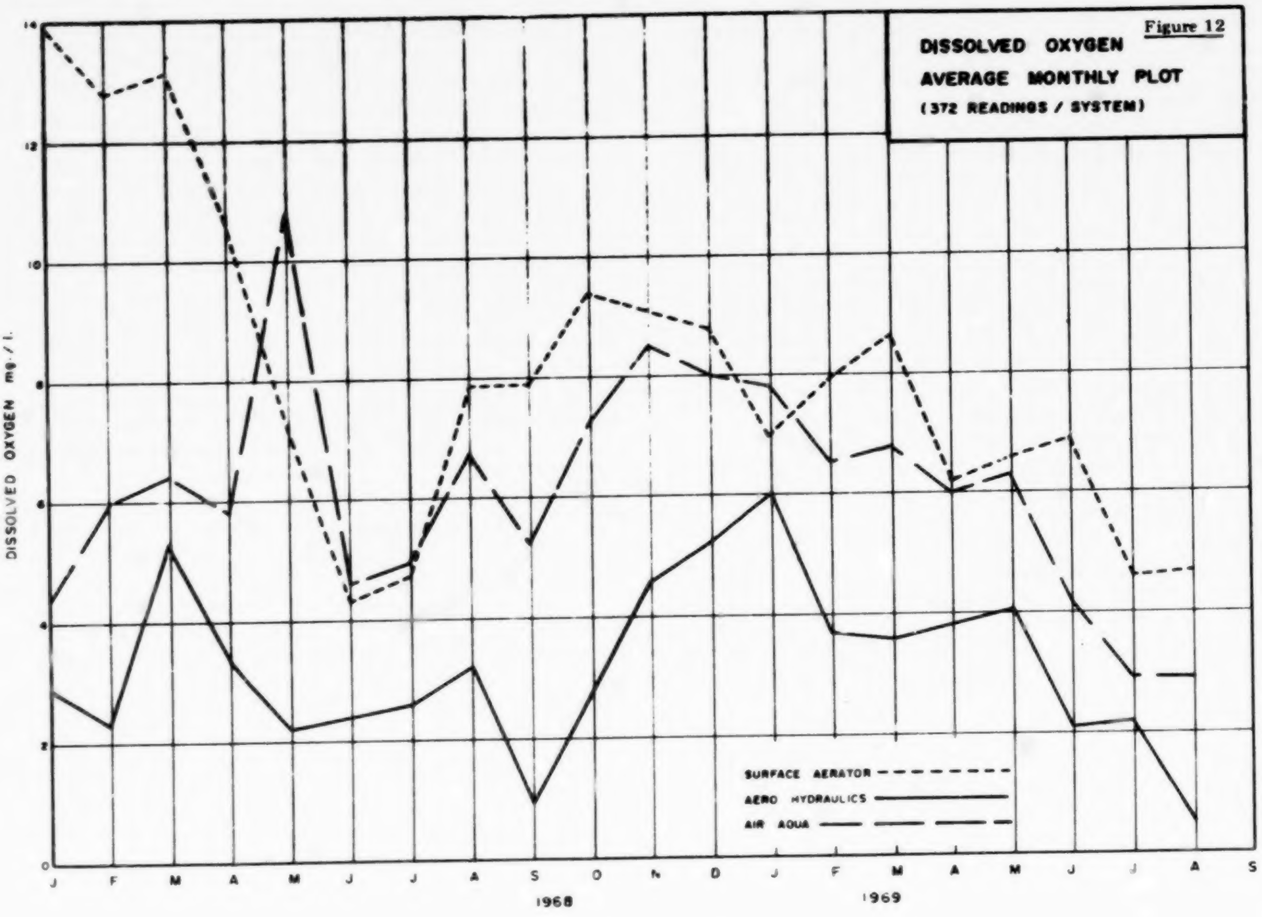


Figure 12

DISSOLVED OXYGEN AVERAGE MONTHLY PLOT (372 READINGS / SYSTEM)

APPENDIX B

OPERATIONAL OBSERVATIONS AND PROBLEMS

Weed Control

No particular problems were encountered with the control of weed growth on the test cell banks. Scheduled grass cutting on the sodded banks and judicious application of weed control chemicals on the rip-rapped sections of the banks were carried out to control this nuisance with good success.

Mosquito Control

Although at times the mosquito population appeared to be large, actual counts taken showed that the lagoon area had one of the lowest mosquito counts of the entire Metropolitan Area. Here it should be noted that the aerated lagoons are surrounded by conventional stabilization ponds, with the area ratio between aerated and conventional cells being in the order of 1:25. Regular spraying by the Mosquito Abatement Department was carried out.

Grease and Scum

Build-up of grease and scum was noted to occur in the corners of the Air-Aqua primary cell, with very little floating material having been noticed on the surface aerator and Air-Gun cells. The Air-Aqua system is designed with a center dike, one of its purposes being to intercept floating material and prevent it from escaping to the effluent.

Odours

In general it may be said that odours emanating from the three systems were of a minor nature and they certainly did not carry beyond the test area. The only exception was the Air-Aqua primary cell which at times released odours that were quite noticeable on the enclosing banks. They were attributed to the floating grease and scum collecting along the banks and in the corners of this cell, where this material would decompose, releasing odours. This occurred primarily along the south banks, and may well be the result of the very flat slope (8:1) incorporated in the design.

Ice

Ice build-up caused the greatest problems with the surface aeration system. The Air-Aqua and Air-Gun systems were notably free of problems due to ice build-up.

The Air-Aqua system did not cover completely with ice for the winter period and showed the peculiar build-up of "stooks" over the air lines as observed in other installations. Ice thickness found on the Air-Aqua secondary cell varied from approximately 6" to 48" from inlet to outlet, respectively. With water depth of 10'0" this will cut down the retention period considerably during the winter when water temperatures are adadverse to the promotion of biological activity. The foregoing is of course true for all lagoon type systems, be they conventional or aerated. No ice thickness surveys were carried out on the remainder of the test cells due to thin ice.

The Air-Gun system retained open water longer than the Air-Aqua. With the onset of colder temperatures the openings immediately above the guns decreased in size from inlet to outlet. Under severe temperature conditions the openings became covered with ice domes towards the outlet end. Open water conditions existed year round above the first two or three rows of guns.

Ice built up on the surface aerators to such an extent that it was impossible to clear the ice from the aerator area. The results was that during the winter only three or four surface aerators could be kept operating.

Foaming

As with conventional lagoons or stabilization ponds, foaming conditions were encountered to some degree with all three aeration systems. The surface aeration system showed more foam than the other two systems, due to the more violent agitation of the surface of the water by the aerators.

The quantity of foam generated varied with the water temperature, the maximum condition occurring after spring break-up when water temperatures were rising. However, the foaming did not reach a point where it became a problem.

Grit and Rags

After one year of operation considerable quantities of grit and rags were present in all three systems. For the effective long-term operation of any aerated system, pre-treatment facilities will be required to remove grit and rags.

Bank Slopes

The following observations on bank slopes were made:

Air-Aqua — The existing 8:1 slope utilized in the Air-Aqua cells created a wide unaerated band around a portion of the lagoon; this likely resulted in some short circuiting. If flat slopes are required for stability, consideration should be given to providing aeration along the slope. The majority of the dikes for the demonstration lagoons were constructed at 3:1; this slope was stable, but for long-term operation 4:1 is recommended. The slopes of the Air-Gun cell were constructed at 2.5:1 to minimize surface area. These slopes proved unstable under draw-down conditions. The rip-rap provided effective erosion control in all cells.

Equipment Operations

a) Air-Aqua system — After start-up, difficulties were encountered with pressure build-up in the system. Recommended HC1 acid gas cleaning of the tube system was performed without alleviating the problem. Removal of 250 foot length of tubing revealed that the perforations had not been properly cut. All tubing was repunched in-situ requiring the lowering of the system so that the workers could walk through the cells feeding the tubing through a punching mechanism mounted on a boat. The repunching took 2 man-days to complete.

Upon completion of this work, pressure on the system dropped to 5.6 to 6 psig. The manufacturer then recommended that acid cleaning be carried out quarterly regardless of pressure build-up.

When operating properly the pattern of air distribution should show a grid of air bubbles, released by the perforated tubing. Although this grid did show immediately after installation and repunching of the tubing, it deteriorated after a period of operation due to clogging of the perforations, regardless of acid cleaning, carried out regularly. This clogging had affected the air distribution pattern and was confirmed by D.O. tests. Further investigations traced the loss of pattern to water in the tubes and condensation and from water coming back through the valves during power failures. By manipulating the acid valves it was possible to force the majority of the water out of the tubes, restoring a reasonably good air pattern.

Supporting equipment such as air-compressors, effluent recirculation pump, flow meters, and samplers were subjected to a regular preventative maintenance program, and few problems were encountered.

b) Surface Aeration system — Ice has been a problem with the operation of the surface aerators. The basic weakness was the ice build-up on the impeller shaft and supporting structure, with subsequent freeze-up and stopping of the mechanism. Ice formed on the piles and impeller, resulting in a limited clearance between the rotating ice on the impeller and the fixed ice on the piles. This is a problem which is difficult to overcome with the winter climate experienced in the Winnipeg area. Two methods aimed at overcoming this problem were attempted; firstly, one of the platforms was shrouded with plywood, and secondly a second unit was shrouded with flexible nylon cloth. Both attempts failed.

Other results of the ice accumulation were off-balance, causing vibration of the supporting structure and misalignment of motor reducer, resulting in a coupling failure and loosening of impeller, bending and loss of blades.

In the spring of 1968 the manufacturer replaced all blades with a heavier design. Although this was thought to cure the problem encountered with the impeller blades, subsequent winter operation disproved this, as bending and loss of blades occurred.

It may be possible to reduce or eliminate the icing problems with a change in the supporting structure. The existing structure with four piles provided a surface for the ice to grow on, a two pile arrangement with the piles widely spaced may be more successful.

c) Air-Gun system — No problems were encountered with the operation of this system until October 1968, when it was noticed that the most northeasterly gun was discharging air continuously. During the winter 1968-69 this condition spread to most guns. It was thought to be caused by the build-up of either ice or rags in the syphon chamber. The latter proved to be true. In May 1969, the manufacturer installed syphon chambers of a new design on 42 guns. The new design having larger clearances, may eliminate the problems of plugging of the syphon chambers with rags. Insufficient time has elapsed since the modification to allow an assessment of their long-term dependability.

Capital Costs

Two sets of cost data have been prepared for aerated lagoons; firstly, the 0.5 Imgd demonstration lagoons at Charleswood and secondly, a general unit capacity cost curve.

All costs and estimates in this report are adjusted to an Engineering News Record Sewage Treatment Plant cost index of 132.0. As costs change the appropriate adjustment must be made to the reported costs.

The capital costs for the Charleswood demonstration lagoons are shown in Table 2. These costs have been arrived at by assuming each system would be as indicated in Figure 1, insofar as floor dimensions and identical equipment layout are concerned. However, in order to relate realistic costs to a permanent installation it is necessary to assume each unit to be constructed as a totally independent unit. An independent unit is one with independent piping, diking, influent-effluent structures and electrical supply. Therefore, only items 1 and 2 of Table 2 are established directly from the contract amounts. The remaining items were arrived at by combining estimated quantities and the actual unit prices tendered. The general contractor for the lagoon project was consulted on the tendered prices. All costs were considered realistic with the exception of rip-rap. In the case of rip-rap, he was of the opinion that the price should be 1.5 times the tendered price (ie. $12.00/cu. yd. in place).

The estimated quantities for earthwork were calculated on the basis of 3 feet freeboard, 12 feet roadway width, and 4:1 dike slopes, rip-rap facing the interior dikes from toe of slope to 1 foot above the normal operating level. The slope estimated is flatter than those on the demonstration project. These adjustments are considered necessary to ensure a permanent and relatively maintenance free structure with the soils encountered in the Winnipeg area.

Sodding and seeding quantities include sodding the interior dikes above the rip-rap and 50 percent of the total dike crest, and seeding the exterior dike slopes. Roadway quantities are based on an asphaltic surface treatment being applied to the unsodded dike crest.

The electrical costs include service entrance equipment, motor starters, and lighting for the equipment and structures associated with the respective systems. Power distribution costs were based on the billing received from Manitoba Hydro for the supply and erection of equipment, and allocation to the respective systems on the basis of length of cable and number of poles required for each.

Actual influent and effluent chamber and piping costs were re-estimated on the basis of piping and chambers being of such length and size for an independent treatment system of 0.5 Imgd capacity. Fencing costs are for the perimeter of the cell taken at the toe of the dike slope.

Table 2

Capital Costs - Charleswood Demonstration Lagoons

	Air-Aqua	Surface Aerator	Air-Gun
1. Aeration Equipment, Supply (Blowers, Headers)	$ 47,500	$ 44,300	$ 50,000[1]
2. Aeration Equipment, Installation (Headers, Housing, Platforms)	23,000	56,500	33,700[2]
3. Electrical	3,500	7,800	3,500
4. Influent Piping	3,200	3,200	3,200
5. Effluent Piping	3,500	2,200	2,200
6. Influent Chamber	17,900	17,900	17,900
7. Effluent Chamber	22,900	22,900	22,900
8. Excavation & Dike Construction	32,400	29,200	27,000
9. Clearing, Grubbing Unsuitable Material	10,800	10,800	10,800
10. Rip-rap	93,000	73,500	70,300
11. Seeding & Sodding	6,400	5,000	4,000
12. Roadway (asphalt)	3,900	3,500	3,200
13. Chlorination Facilities (including housing)	20,000	20,000	20,000
14. Power (hydro)	3,500	5,400	3,500
15. Instrumentation (including magnetic flow meter)	10,000	10,000	10,000
16. Fencing	4,700	4,000	3,500
Total Estimated Cost	$306,200	$316,200	$273,700
10% Eng. & Contingencies	30,600	31,600	27,300
Total Estimated Cost	$336,800	$347,800	$301,000

Costs indexed to ENR S.T.P. cost index 132.0

1) includes installation, excludes blowers and header

2) includes blower and header

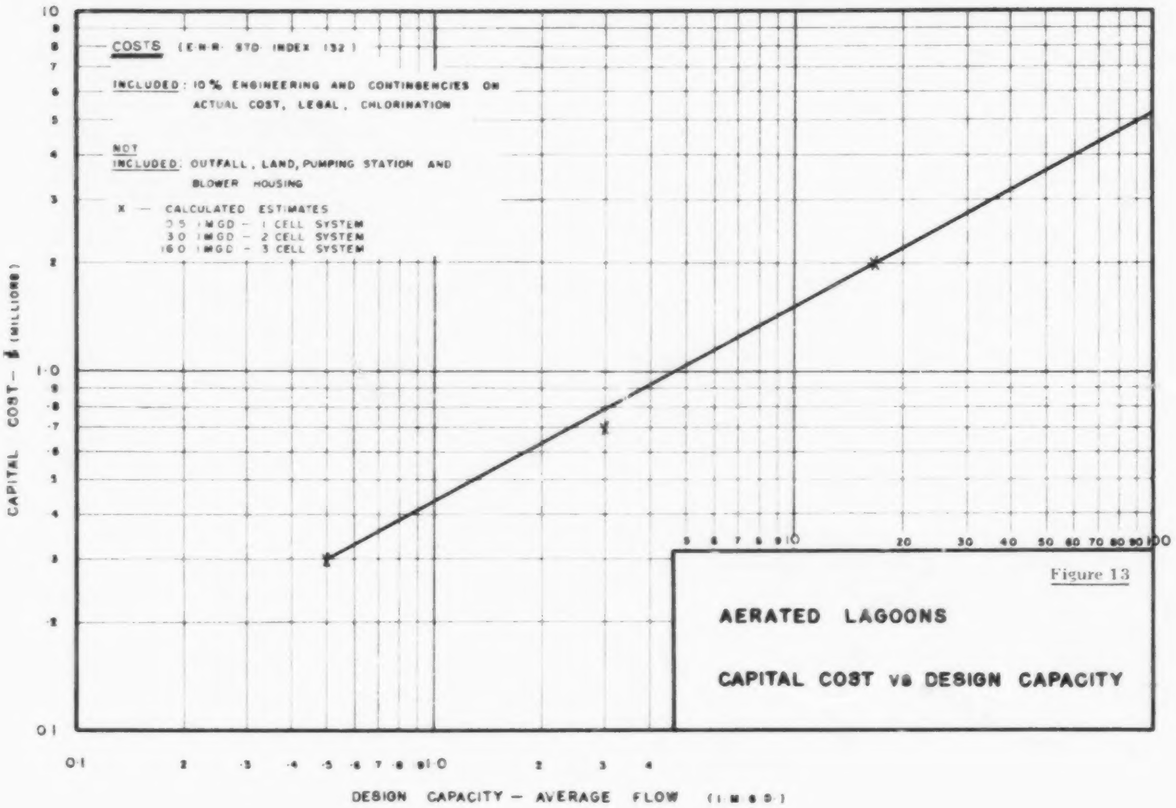


Figure 13

AERATED LAGOONS

CAPITAL COST vs DESIGN CAPACITY

Additional items of chlorination facilities and instrumentation are included since these would be desirable in a permanent installation. The costs do not include pumping station, forcemain, outfall, or land. However, they are inclusive of engineering, legal and administration charges.

From Table 2 it can be seen that the total costs for the Air-Gun are slightly lower than the Air-Aqua and Surface Aerator Systems. Major costs differences are due to additional rip-rap and excavation required for the dividing dike, greater dike perimeter for the Air-Aqua System, and platform costs for the surface aerator.

Figure 13 shows the development cost of the aerated lagoon process per unit of lagoon capacity. These costs have been developed by applying the unit material and installation costs of the demonstration lagoons to calculated quantities. Equipment supply and installation costs were derived from equipment manufacturers' quotations for similar facilities at other treatment works.

Operation and Maintenance Costs

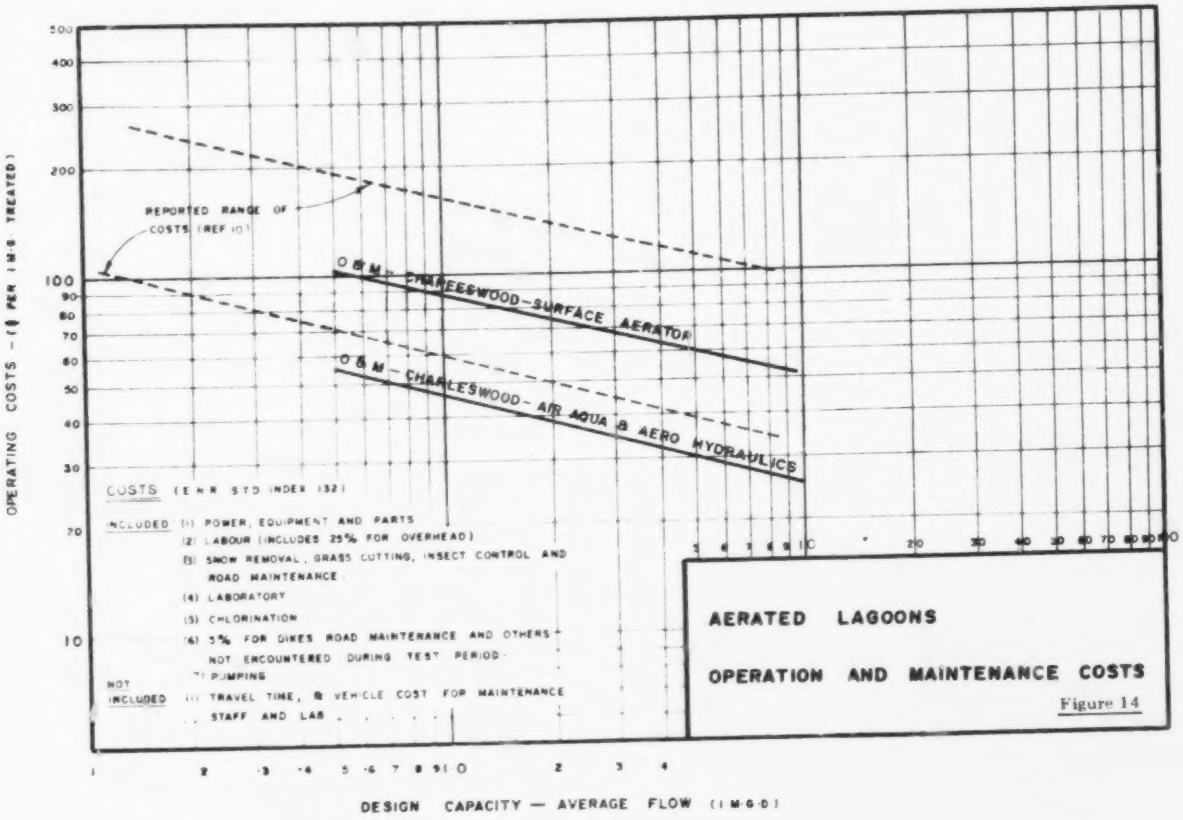
The operating and maintenance costs for the 0.5 Imgd Charleswood Demonstration Lagoons are shown on Table 3. They are based on 17 months of operation.

Assuming chlorination of the effluent at 8 mg/l is desirable, the total operating costs should be adjusted to $53, $107 and $56 for the respective lagoons.

The power costs tabulated in Table 6 are based on field measurements of true power drawn with a unit power cost of 0.9¢/kwh. Allocated to each system is a share of the heating and sampling pump load. In the case of the surface aeration the cost is calculated on all eight units operating. In terms of oxygen requirements, the surface aerators are capable of treating the excess of the rated flow. However, at the time of design eight units were considered necessary to ensure adequate mixing.

Repairs include regular weekly inspection and servicing of equipment, non-routine repairs and replacement parts were required. The cost of repunching the aeration tubes and replacing the siphons on the Air-Guns is not included in Table 3. The labour and materials on the servicing were based on actual time card and material receipts and allocated to the respective systems. Labour includes a 25 percent factor for overhead. General costs include maintenance to the common inlet chamber, metering equipment, etc. Since it was difficult to allocate these costs to a specific lagoon they were split equally to the three systems. Laboratory costs include labour and supplies for sampling and analysis. Actual laboratory costs were three times the cost reported due to the frequency and number of analysis performed under test conditions. Therefore, the lower costing for laboratory reported here would be a more realistic value under normal operating conditions. Maintenance and laboratory personnel were not based at the lagoons during the test therefore considerable labour and vehicle time for traveling was not charged to the test lagoons.

The development of operating and maintenance costs for aerated lagoons is shown in Figure 14. The curve was prepared by projecting the actual Charleswood demonstration lagoon costs on the trend line as indicated.



AERATED LAGOONS

OPERATION AND MAINTENANCE COSTS

Figure 14

Table 3
Actual Operating and Maintenance Costs
0.5 Imgd Demonstration Lagoons

	Avg. Cost per Img Treated		
	Air-Aqua	Surface Aerator	Air-Gun
Power	$ 10	$ 64	$ 15
Equipment Servicing & Repair	13	14*	12
Laboratory & Control	13	13	13
Road & Dike Maintenance	4	3	3
Snow Removal, Grass Cutting, Mosquito Control	3	3	3
General	3	3	3
Total Operating & Maintenance Costs per Imgd Treated	$ 46	$ 100	$ 49

* not continuously maintained due to ice.

The slope of the line was guided by costs reported in recent literature (6). The reported costs from the literature reference have been adjusted to Imperial Gallons and are also shown on Figure 16. It is noteworthy that the referenced costs are representative of power costs of 1¢ per kw. hr. and labour of $5.00/hr., while in comparison the local power and labour costs are approximately 0.90¢ per kw. hr., and $4.25 per hr., respectively. Very little cost information at other aerated lagoons is available to confirm the trend of costs for increasing capacities. However, a check was made on the expected operating and maintenance costs by determining the future power requirement for mechanical equipment coupled with current power rates. The total costs were extrapolated from the power costs on the basis of power being about one-third of the total costs. This provided a reasonable check on the trend developed by the literature.

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BIO-FLOTATION TREATMENT OF REFINERY WASTEWATER EFFLUENTS

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INTRODUCTION

Located on the southwest shore of Lake Michigan, American Oil Company's refinery at Whiting, Indiana, has a crude running capacity of 290,000 barrels a day and produces a complete line of petroleum products. Process and cooling water is pumped from Lake Michigan at a maximum rate of about 170 million gallons a day and is returned to the lake after treatment. Most of the water is used only for cooling. Cooling water is handled in a separate collecting system and gravity separator. An additional 100 million gallons a day of cooling water is recirculated through cooling towers and reused.

All sanitary sewage is routed to a municipal treatment plant--none enters the lake in refinery effluents.

The secondary treatment of oily process water is the subject of this paper; however, to put the subject in proper context a limited description of the peripheral treatment will also be made.

REFINERY EFFLUENT TREATMENT PRIOR TO JUNE, 1967

Prior to about June, 1967, effluent water treating facilities at the Whiting refinery were as shown in the first figure. These facilities continue to be a part of the present effluent quality control system.

Cooling water is treated in the API separator shown at the bottom of Figure 1.

About 30 million gallons a day of spent process water is carried from the refinery through a large underground collecting system and enters the modern oil-water separator, shown in the center of the figure, through a bar screen.

The water is then distributed among the compartments of the oil-water separator. Each of the 34 compartments consists of a primary and secondary stage. Twenty-four of the 34 compartments are used for primary oil-water separation which provide about 90 minutes of retention time at present flow rates. The compartments are equipped with mechanical flight scrapers to recover the separated oil from the surface and remove separated solids from the floor of each compartment.

Oil and emulsion is removed from the surface by gravity to large rectangular basins shown located near the center of the separator.

From these collecting basins, oil and emulsion are transfered to emulsion treating tanks shown at the left. Oil from emulsion treating is returned to the refinery; water is returned to the process water separator. Sludge, which is removed from the bottom of the separator compartments, is transferred to the vertical tank at the right of the process water separator and processed through a centrifuge mounted under the tank.

Effluent from the process water separator is lifted by pumps located in the adjacent two-story building and discharged into a concrete flume, which carries it to the lagoon shown in the upper right portion of the slide. Water enters the lower left-hand compartment through a distribution baffle. The first compartment provides about four hours of additional gravity settling. Facilities are available for periodic removal of oil from this first presettling compartment of the bio-unit. The water passes from the presettling compartment, under oil retention baffles, over control weirs, and through Passavant brush-type mechanical aerators as it enters each of four aeration compartments in series. The retention time in each aeration stage is about four hours.

Biological oxidation, flocculation, and air flotation of flocculated biological slime and occluded oil occur in each aerated compartment, hence, the designation Bio-Flotation. Surface skimmings are removed from each compartment and, prior to the recent startup of tertiary treatment facilities, effluent from the Bio-Flotation unit was carried through a second concrete flume and enters the lake at a point located in the lower right-hand corner of the slide.

Figure 1 - Effluent Water Treating Facilities at American's Whiting, Indiana, Refinery Prior to 1967.

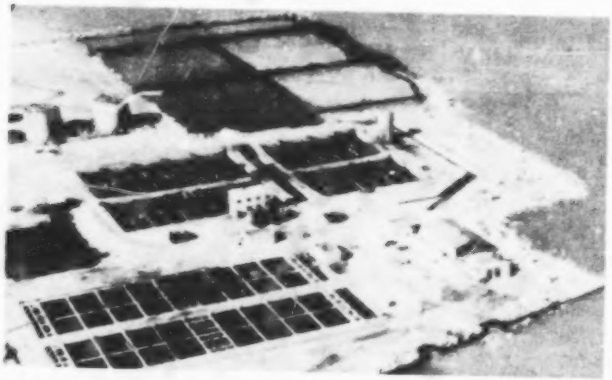


Figure 2 - Delta-Brush Aerators

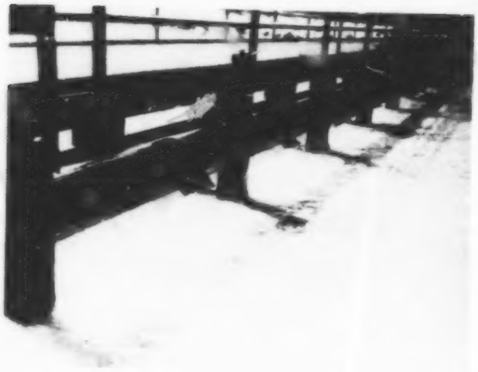
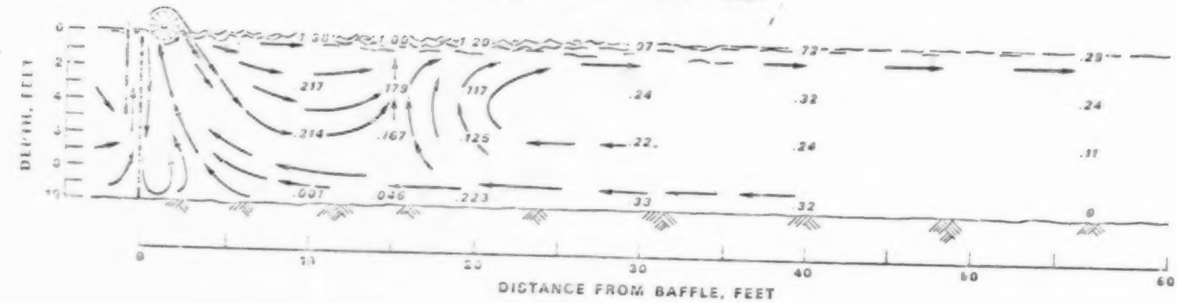


Figure 3. Current Direction and velocities in aeration compartment

NUMBERS IN ITALICS INDICATE WATER VELOCITY, FT./SEC.
BRUSH: 120 RPM
BRUSH BAFFLE: NOT SUBMERGED
THEORETICAL VELOCITY~0.0326 FT./SEC.



A section of the brush-type aerators, which were imported from Germany, as shown in Figure 2. These are the "Delta-Brush" modification and have a diameter of about 20 inches. Approximately 720 linear feet of these units supply about 25,000 pounds a day of dissolved oxygen to the process. Note the milky appearance of the water due to the large amount of entrained microbubbles. It looks like–and acts like–dissolved air flotation.

The hood, or baffle, associated with the Delta-Brush as shown in Figure 3 serves a number of purposes. According to the work of Pasveer in Holland, a baffle having an angle of 52° with the water surface increases oxygen transfer significantly; it may be used to limit turbulence downstream; and it may be used to control somewhat the direction of exiting water. If the baffle ends above the surface the water exits along the surface, but the bottom of the baffle also can be submerged to direct the flow of water and entrained air downward.

The Delta-Brush aerator pumps large quantities of water and, by the circulation pattern established in the first 25% of each aeration compartment, biological solids are internally recycled at the inlet end of each compartment. In the body of water the volatile suspended solids concentration equilibrates at 20 to 40 mg/1. Note that this system is certainly not completely mixed. The Delta-Brushes only provide mixing in the initial 25% of each aeration basin, or about the first 70 ft. of the 275 ft. long basin.

The volatile suspended solids generated in excess of the 20 to 40 mg/1 equilibrium level amounts to about 15,000 lbs./D. These are removed from the system by bio-flocculation. As the slime particles form, they serve as nuclie for coalescing oil and entrained matter, and at the same time occlude microbubbles of air generated by the Delta-Brush. Occlusion of air and oil makes the slime particles sufficiently buoyant so that they float to the surface in the downstream quiescent section of each Bio-Flotation compartment, from whence they can be removed by skimming. This not only decreases treatment costs because the comparatively expensive process of biochemically oxidizing the flocculated materials is avoided by their physical removal, but extended pilot plant work

Table 1. Bio-flotation treatment of refinery process water

	30 MGD	PRE-SETTLING BASIN	1ST AERATION BASIN	2ND AERATION BASIN	3RD AERATION BASIN	4TH AERATION BASIN
OIL, MG/L	150	100	75	45	25	20
TOTAL SUSP. SOLIDS, MG/L	56		39	60	32	34
VOL. SUSP. SOLIDS, MG/L	46		20	34	19	26
BIO-FLOATED SLUDGE						
OIL/SOLIDS RATIO	—		2.5	2	1	.5
LBS. OIL	—		6,250	7,500	5,000	1,250
LBS. SOLIDS	—		2,500	3,750	5,000	2,500
TOTAL CONTAMINANTS	—		8,750	11,250	10,000	3,750
O_2 UTILIZATION						
PPM/HR.	.6		5.43	14.3	9	4.8
PPM/PPM VSS/D	—		6.48	10.1	11.4	4.4

demonstrated the continuous physical removal of the bio-flocculated material resulted in a significant increase in the degree of purification attained. The TON, a particularly refractory pollution parameter in refinery waste water, was especially aided by the continuous physical removal of the bio-flocculated material.

The physical removal of contaminants by Bio-Flotation is illustrated in the material balance shown in Table 1. Even though the process water receives about 1½ hours sedimentation in the API separator, the water is a combined waste stream from an extremely diversified refinery and because of the various emulsifiers, the effluent from primary treatment benefits from further gravity settling. Pumping the effluent from primary treatment to the aerated lagoon via a flume causes a significant flocculation such that about a 35% reduction in oxygen demand, oil content, and solids occurs simply by supplying another four hours sedimentation. This contaminant reduction, of course, is accompanied by sludge deposition and accumulation of floating, agglomerated oily emulsion. The solids up to this point are comparatively inert as indicated by the oxygen utilization rate. The solids in the vicinity of the brush in each succeeding aeration basin, however, increase rapidly in activity until the food supply is limited, as in the front end of 4th basin, and the activity decreases. The activity of the solids in the effluent is about 1/10 th that of the 4th aeration basin.

As the active biomass increases in each aeration compartment, the amount in excess of that required to maintain a 20 to 40 ppm vss in the dispersed phase flocculates the emulsified oil and solid contaminants into large agglomerates most of which occlude air, float, and can be removed by skimming. Thus, bio-flocculated contaminants can be removed from each aeration basin. The composition of the bio-flocculated sludge varies from an oil/solids ratio as high as 2.5/1 in the 1st aeration basin rich in oily contaminants, to an oil/solids ratio as low as 1:2 in the final aeration basin lean in oily solids. If the air floated bio-flocculated sludge is skimmed carefully, the water content will average about 70%. In this material balance, the bio-flocculation operation removes about 20,000 lbs. emulsified oil/day as well as large reductions in BOD, COD, threshold odor number, etc. Thus, in excess of 100,000 lbs./D of bio-flocculated sludge is removed.

In contrast to conventional activated sludge treatment in which flocculated biological solids are present at concentrations of the order of 4,000 to 6,000 parts per million, the Bio-Flotation process is characterized by much lower concentrations of nonflocculated biological growths which remain dispersed throughout the body of water and yield oxidation rates per unit of suspended biological solids in the dispersed phase far higher than those reported in the literature for the activated sludge process.

Recent Bio-Flotation Process Improvements

The problem of sludge deposition and resulting adverse effects upon effluent quality is common to all aerated lagoon systems and historically the Bio-Flotation process has been no exception. Each basin of the Bio-Flotation process has an aera slightly over one acre. When the unit was constructed some experimental skimming equipment for removing the bio-flocculated solids was included. This skimming equipment proved inadequate and, through the years, much of the air floated bio-flocculated sludge, de-aerated, or was otherwise dispersed and accumulated as bottom sludge.

The adverse effects of anaerobic digestion of bottoms sludges can be partially overcome by increasing oxygen input, but sooner or later these deposits must be removed.

Four additional 20 horsepower aerators were installed in one aeration basin of the bio-pond in 1967. These units were of the floating pump type as shown in Figure 4. In addition to oxygen, these aerators also supply turbulence to minimize sludge deposition. However, the bio-sludge in excess of the 20 to 40 ppm that remains dispersed throughout the system agglomerates into large sludge particles that resist strongly redispersion or fragmentation.

Figure 4 - Floating Pump Type Aerators

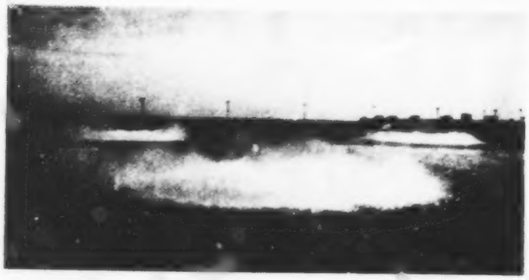
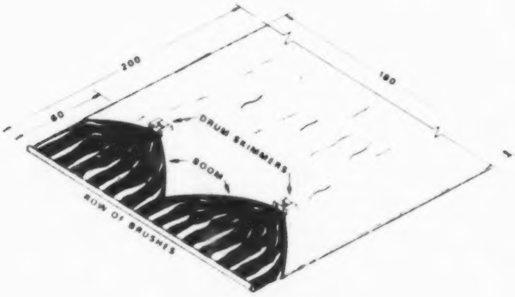


Figure 5 - Flotation with Brushes and Brush Skimmers



Thus, in parts of the basin where velocity currents are low, such as the region where the hydraulic curtain forms between aerators, sludge continued to deposit on the bottom. Further, the oily, bio-flocculated sludge appears to require much higher current velocities than the 0.5 fps accepted for maintaining the dispersal of activated sludge.

A number of improvements are being made to the Bio-Flotation process in view to preventing sludge deposition and developing the full potential of the process.

The means developed for concentration and removal of the bio-flocculated material are shown in Figure 5 and consist of a combination of brush aerator/boom/drum skimmers arranged in a fashion that describes a hydraulic regime making the system operable. The *brush aerator* provides for (1) oxygenation, (2) mixing, (3) fine bubbles for air flotation of the bio-flocculated material, and (4) imparting a velocity vector to the water mixture. The *booms* (or baffles) (1) contain the air floated solids, (2) direct the contained solids to the rotating drum skimming device, (3) define the boundary of the system such that the current velocity is adequate for directing the air floated solids to the drum skimmer, and (4) direct surface currents to provide enough hydraulic pressure at the drum skimmer such that any large size accretions of air floated solids are deformed and pushed against the surface of the drum which facilitates the skimming efficiency of the drum in this service. The *roller drum skimmers* provide (1) a means for removing the air floated bio-flocculated material continuously with limited operating attention and (2) for removal of the surface material in a dewatered, prethickened state. Two views of the operating facility are shown in Figures 6 and 7.

The water velocities are important for functioning of the skimming equipment. If the current is more than 3 fps at the containment baffle too much of the air floated bio-flocculated sludge escapes. Moreover, surface currents at the drum skimmer must be at least 0.3 fps to overcome wind effects and the propensity of the air floated bio-flocculated sludge to bridge and not move into the skimmer as shown in Figure 8.

Further, the air floated bio-flocculated sludge rapidly agglomerates into a mass too weighty for the modest cohesive forces between the drum and sludge to overcome as illustrated in Figure 9. Consequently, the drum cannot lift the oily sludge mass from the surface of the water if the size of the sludge mass is not controlled. Fortunately, surface currents can be used to overcome bridging and push the sludge mass onto the drum surface, assuring contact, simultaneously deforming agglomerated sludge into a smaller, more fluid mass that the drum skimmer is capable of handling. In practice, the drum skimmers have frequently been overloaded and more effective skimming equipment is being developed.

Earlier I mentioned that the Delta Brushes only provided mixing in the first quarter of each aeration basin. Therefore, some of the air floated bio-flocculated sludge that escapes the brush/boom/drum skimmer facilities will de-aerate and separate as bottoms sludge in the downstream portion of each aeration basin.

A device for keeping biological solids in suspension, providing additional control over the rate of sludge deposition, and increasing the oxygenation capacity of brush-type aerators is shown schematically in Figure 10. The pumping action of pontoon-mounted brush-type aerators, which have been equipped with submerged baffles, causes them to orbit around a tether post. The submerged baffles provide a means of restricting the volume of water under the immediate influence of the aerators, increasing oxygenation efficiency, controlling turbulence, and offer a degree of process control. The bypass baffle is adjustable such that the mass of water discharged by the aerator can be varied from essentailly total confinement within the baffled zone to total discharge. When totally discharged the mixing energy of the orbiting aerator is dispersed to the lagoon proper. This maximizes dispersion of solids throughout the lagoon and maximizes carry out of solids.

On the other hand, if the mixing energy is partially confined to the baffled zone, the active bio-solids inventory in the basin increases as bottom sludge deposits are lifted, aerated, and allowed to partially resettle during each revolution of the unit, which requires about six minutes under present operating conditions.

By orbiting around a tether, this particular unit will sweep a section of one compartment of the Bio-Flotation basin more than 100 feet in diameter as shown in Figure 11. If mechanical performance of the prototype proves fully satisfactory, additional baffled, orbiting aerators will be installed in other compartments. It is anticipated that

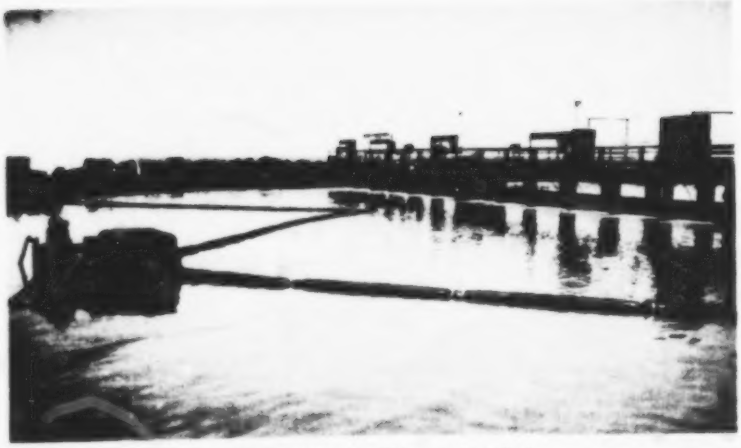


Figure 6 - Brush/Boom/Skimmer Installed

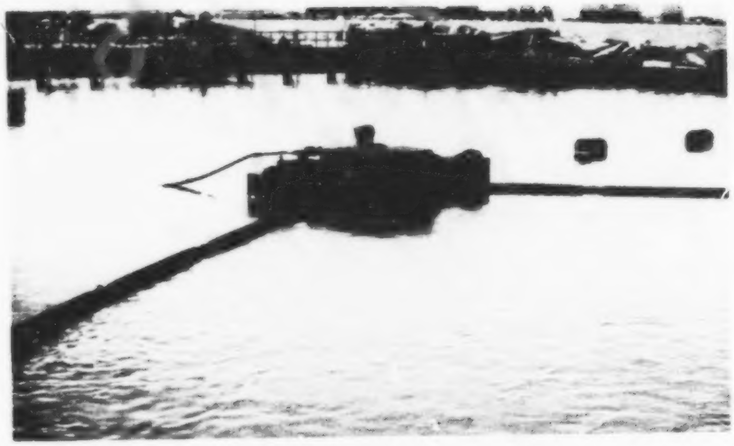


Figure 7 - Roller Drum Skimmer Showing Air Floated Bio-Flocculated Sludge Being REmoved

Figure 8 - Tendency to Bridge REquires That Floating Sludge Be Delivered to Drum Skimmer in a Positive Manner

Figure 9 - Drum Skimmer Cannot Lift Agglomerated Sludge until Fludized by Surface Currents

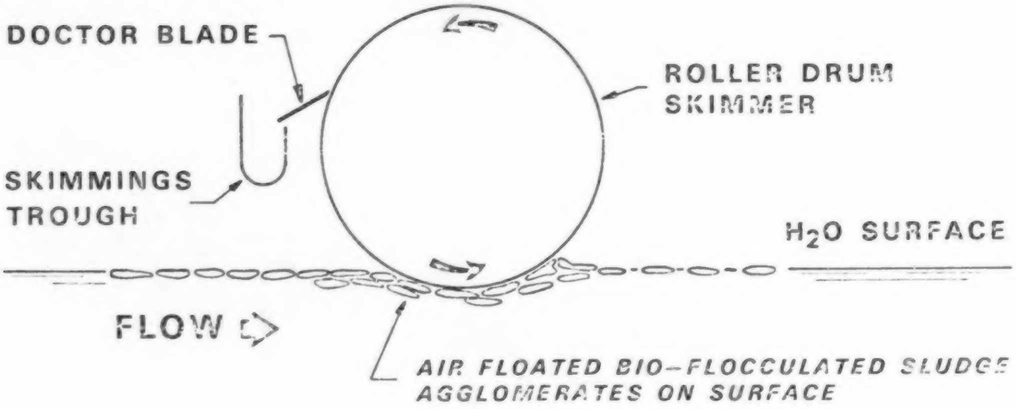

DOCTOR BLADE
ROLLER DRUM SKIMMER
SKIMMINGS TROUGH
H_2O SURFACE
FLOW
AIR FLOATED BIO-FLOCCULATED SLUDGE AGGLOMERATES ON SURFACE

Figure 10 - Sketch of Orbiting Aerator

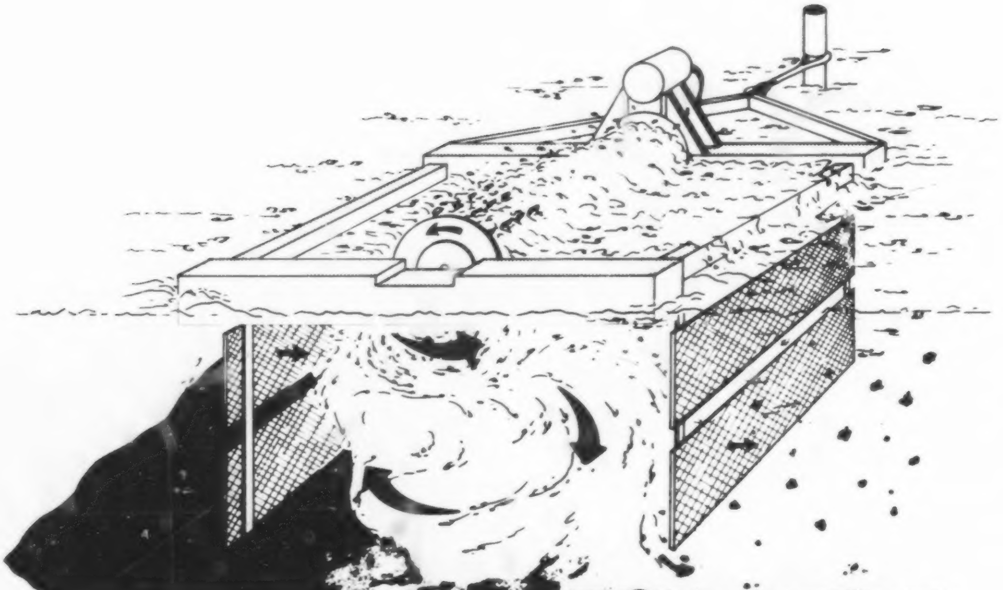


Figure 11. Aeration compartment of bioflotation unit showing location of pontoon mounted brushes to combat deposition of sludge.

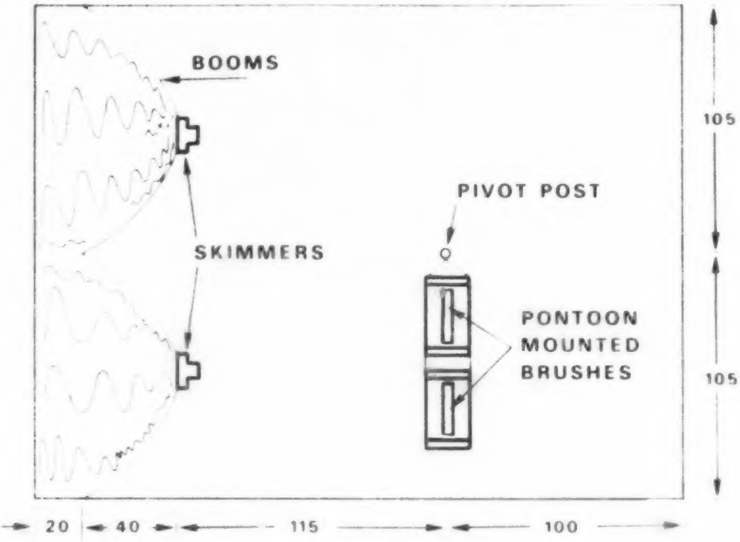

BOOMS
SKIMMERS
PIVOT POST
PONTOON
MOUNTED
BRUSHES
105
105
20
40
115
100

Figure 12 - Orbitting Aerator Installed

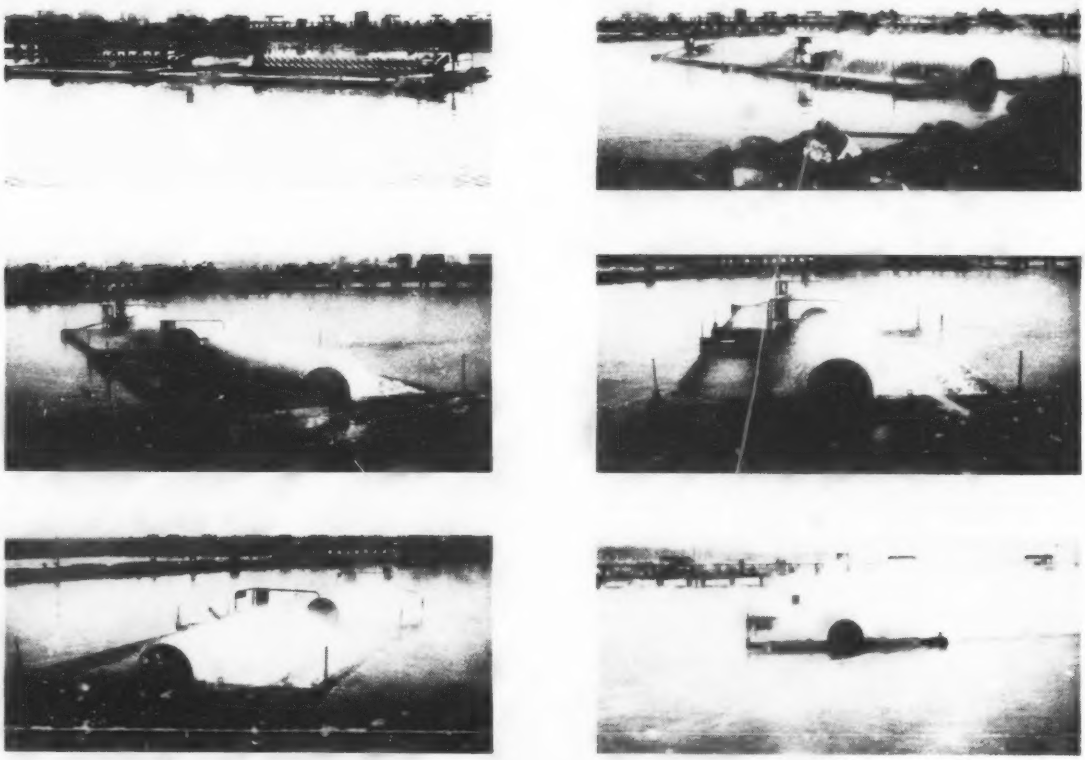


Table 2. Process water treatment

	PILOT PLANT FEED	AIR FLOTATION	MIXED MEDIA FILTRATION	TARGET
TURBIDITY, JACKSON UNITS	50–250	3–7	1–3	7
OIL, PPM (INFRA RED)	20–100	5	5	< 10
SUSPENDED SOLIDS, PPM	20–100	< 10	< 10	< 10
BOD, PPM	20–80	10–20	10–20	< 20
PHENOLICS, PPM	0.2–1 .0	< .1	< .1	< .1
THRESHOLD ODOR NO., % REDUCTION		> 80%	> 80%	> 80%
PHOSPHATE	1–2	< .2	< .2	< .2
CHEMICAL USAGE				
ALUM, PPM		35	15	
POLYELECTROLYTE, PPM		0.2	0	

this development will enable a degree of process control hitherto unattainable. The series of pictures in Figure 12 were taken as the prototype unit passes a point on shore. Note how well the turbulence is confined to the baffled region. In my earlier comments I described the aeration burshes and referred to them as the "Delta-Brush" modification. This unit is another type of rotor, called a Mammoth Rotor, and has twice the diameter of the Delta-Brush and no hooded baffle. The Mammoth Rotor does not generate the microbubbles for flotation as does the Delta-Brush configuration. The final picture in the sequence was taken during winter. While a large amount of water vapor was generated, there was no problem with ice.

Historically, using inadequate skimming and sludge control facilities, the Bio-Flotation process, with 20 hours total retention time, yields an effluent comparable to that obtained from a 3 to 5 day aerated lagoon. Oil, BOD, phenolics reductions range from 60 to 90%.

With complete installation of drum skimmers and orbiting aerators, it is anticipated that consistent water quality will be obtained more comparable to that of an activated sludge plant.

Tertiary Treatment

We have found also that the Bio-Flotation process serves as an effective conditioning step for tertiary treatment by chemical coagulation and phase separation. In order to complete the picutre of future process water effluent treatment at Whiting and to indicate the quality of effluent attainable thru the use of aerated lagoons, our work on tertiary treatment will be described briefly.

As mentioned before, the use of brush aeration results in a highly dispersed, nonflocculated biomass. The effluent from the Bio-Flotation unit contains 20 to 40 ppm biomass that resists further natural flocculation. By coagulating and removing this residual biomass, essentially all the residual contamination can be removed from the effluent.

Pilot plant work has shown that either of two treatment sequences will provide essentially complete removal of contaminants from a refinery process water effluent. Both have involved use of the Bio-Flotation process as a conditioning step, although other forms of aerated lagoons also appear to be applicable, followed by chemical coagulation of suspended matter and other substances, using alum or ferric sulfate as a coagulant. The two processes differ only in the method of phase separation after chemical treatment. One involves air flotation, the other mixed media filtration.

Typical pilot plant data with previously established targets for quality of the treated effluent are shown in Table 2. During the experimental work, by selecting the source of feed it was possible to vary feed quality over a wide range. Within the limits of feed quality shown, it was possible to achieve the target values listed.

Our target of 7 JTU could be met by both processes; mixed media filtration gave somewhat lower turbidities at lower chemical usage.

Both air flotation and mixed media filtration consistently reduced the oil content to about 5 ppm by I.R. analysis. There was no visible evidence of oil in the effluent.

Suspended solids were reduced to 5 to 8 ppm by air flotation and values of the order of 1 ppm were consistently attained by mixed media filtration.

One welcome result is the very large reduction in threshold odor number obtained by both operations.

Phosphate reductions of 80 to 90% were observed due to the usage of alum.

The Fluidized Bed Incinerator

Turning now very briefly to the subject of sludge handling and disposal, current plans are to construct a fluidized bed incinerator to dispose of these and other liquid wastes. Oily primary, secondary, and tertiary water treating sludges in addition to other refinery sludges will require an incinerator to handle 1 to 2 x 10^9 BTU/D heat release depending on the amount of thickening possible. The design of the incinerator will be based on the design of the prototype in operation at our Mandan refinery shown in Figure 13. This unit is completely automated and operates unattended. In fact, the instrumentation is in the control room of an adjacent refinery unit.

With reference to Figure 14, to put the unit into operation the air compressor is started. The air preheater is fired, and as the system is warmed, sand is introduced,

Figure 13 - Fluid Bed Incinerator

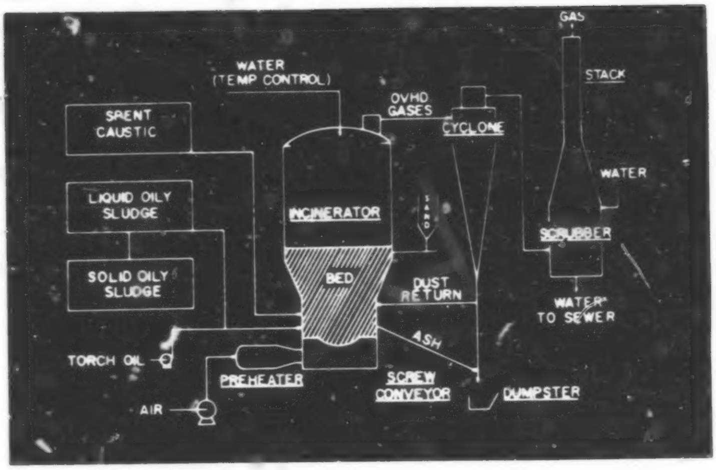

GAS
STACK
WATER
(TEMP CONTROL)
OVHD
GASES
CYCLONE
SPENT
CAUSTIC
WATER
LIQUID OILY
SLUDGE
INCINERATOR
SAND
SCRUBBER
SOLID OILY
SLUDGE
BED
DUST
RETURN
WATER
TO SEWER
ASH
TORCH OIL
PREHEATER
SCREW
CONVEYOR
DUMPSTER
AIR

Figure 14 - Incinerator Flow Diagram

forming a bed of fluidized solids. When sufficient sand has been introduced and the fluid bed is up to 900°F, torch oil is admitted. The burning of torch oil causes a rapid rise in bed temperature. When the bed reaches 1300°F, liquid oily sludge is introduced. Provided the oily sludge contains a sufficient quantity of combustible materials, air heating and torch oil injection can be terminated. The temperature of the bed can then be controlled by balancing feed rate and temperature control water.

As the sludge is burned, the solids remain in the bed, while the gaseous products of combustion and water vapor discharge through the gas cleaning system. The proper bed level is maintained by withdrawing ash as it accumulates from the deposition of solids. Ultimately, the original sand bed is completely displaced by the incoming solids and chemicals. The overhead gases pass through a cyclone separator from which recovered fines can be either returned to the bed or discharged along with the excess bed material. From the cyclone separator, the gases are passed through a water scrubber before being admitted to the atmosphere.

Studies with this incinerator continue in an effort to determine optimum operating conditions and to evaluate operating and maintenance costs. However, one can conclude from experience gained thus far, that the fluid bed incinerator presents a practical solution to the oily sludge disposal problem.

CURRENT DESIGN CRITERIA FOR AERATED LAGOONS

Presented by
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A. *Background*

Conventional lagoons are generally considered to be an aerobic treatment facility, meaning that the biological processes are carried out in the presence of freely available dissolved oxygen. We all acknowledge that settled organic solids undergo anaerobic decomposition yet we normally, but wrongly, consider the lagoon or "oxidation pond" as an aerobic treatment system and not as a facultative device.

Conventional lagoons are solely dependent on nature for the two basic oxygenation processes, i.e. surface reaeration and photosynthetic oxygenation by the algal populations. During the past 10 years the innovation of aerators has been applied to give nature a boost.

Use of these aeration devices was initially limited to alleviating existing problems in overloaded lagoons thereby improving effluent quality. A bonus was discovered that due to the increased turbulence and turbidity, and other factors, normal and often excessive algae growths were retarded in aerated lagoon facilities. Also aerated lagoon facilities could provide satisfactory BOD removals with sharply increased loadings over a shorter detention time than the conventional lagoon. These factors contributed significantly to the birth of the original design of aerated lagoons. Increased demands for more economical utilization of available land for waste water treatment has also stimulated the development of the aerated lagoon concept.

The uncomplete mixed aerated lagoon is a facultative treatment system since turbulence levels are not maintained high enough to maintain solids in suspension. Design usually is for enough power input to create a turbulence level sufficient to insure dissolved oxygen dispersion throughout the liquid. The majority of solids settle to the lagoon bottom where they undergo anaerobic decomposition.

B. *Criteria*

Current design guides or criteria for aerated lagoons are both abundant yet scarce or nonexistent depending upon the desired source. Manufacturers of aeration equipment, both mechanical surface or diffused units, can present the engineer, municipality, industry or state regulatory agency a dazzling array of operational and laboratory test data to support their individual equipment's aeration and mixing capabilities.

Technical research papers have pretty thoroughly covered the field of design theory for aerated lagoons. These papers contain design equations for the biological functioning of an aerated lagoon. Since BOD removal in an aerated lagoon system is primarily a function of detention time, temperature and the nature of the waste, proper application of these rational and empirical equations provides the means to accurately design aerated lagoons.

Published design criteria or standards by state or regulatory commissions are scarce. My correspondence with these agencies revealed that a few have "in house" standards for review purposes by their staffs or that current studies were in progress to prepare an official design criteria. Most agencies indicated that each application is individually reviewed.

The varied applications for aerated lagoon treatment compound the problem of preparation of a design criteria by a state or regulatory agency. Consider the following:

1. Pretreatment of industrial wastes.
2. Total treatment for combined domestic and/or industrial wastes.
3. Total treatment for combined domestic and/or industrial wastes followed by settling and chlorination, etc.
4. Tertiary or polishing of secondary plant effluent with or without further settling, chlorination, etc.

Therefore, the promulgation of standards will be a significant task if the criteria are to be complete. Each of the varied uses of aerated lagoons must be individually considered.

Ohio has a published policy for aerated lagoons for treatment of domestic wastes. The following paragraphs are excerpted form this:

"Aerated lagoons should have a water surface area of one acre for each 1,000 design population. When the BOD of the waste is more than 200 mg/1, the water surface should be one acre per 167 pounds of BOD. Where aerated lagoons are used for secondary treatment following a conventional primary treatment plant, the lagoon area should be 67% of the area required for complete treatment. Where aerated lagoons are used as tertiary or polishing lagoons after an activated sludge treatment process, the lagoon area should be 10% of the area required for complete treatment. After extended aeration treatment, the lagoon area should be 15% of the area required for complete treatment. If aerated tertiary treatment lagoons are proposed after trickling filter treatment, the Ohio Department of Health should be contacted before the plans are prepared. All air should be applied to the first approximate 2/3 of the lagoon area with the remaining 1/3 used for settling."

"Because aerated lagoons are just beginning to be considered in Ohio for waste treatment, the Ohio Department of Health cannot advise regarding air requirements based on experience. It is believed 1,000 cu. ft. of air per pound of BOD will normally be adequate. In every case consideration should be given to the installation of additional facilities if they are found to be needed. Mechanical aerators should be designed on a basis of approximately 1.6 pounds of oxygen transfer per horsepower hour. Mechanical aerators should be properly spaced to provide uniform aeration. Standby aeration capacity should be provided equal to the capacity of the largest proposed aeration device."

Note that no mention or consideration of depth has been made in the loading parameters. However, in all fairness, their criteria notes that every project will be considered individually by the Ohio Department of Health.

The question now arises as to whether or not we need design criteria. Granted that construction standards are useful from the health and safety aspect, e.g. fencing, warning signs, provisions for flow metering, dike slopes, percolation rates, access, etc. However, restrictive design standards are an affront to the professional capability and judgment of the designing engineer.

We have all experienced the rational development of design criteria for conventional lagoons which in numerous instances are so provincial that development and innovations to this technique of treatment have been squelched.

Would it not be better if the Engineer presented the reviewing agency complete background, design theory and equipment capability data for individual project consideration rather than agencies promulgating individual design criteria for aerated lagoons?

Since it is inevitable that regulatory agencies will eventually produce design criteria or standards for aerated lagoons, the following points are offered for consideration:

1. Loading parameters should be expressed in pounds per unit volume, not area, since operating depths of aerated lagoons vary widely.
2. Detention times should be expressed as a function of temperature. Allowances should be encouraged in design for summer vs. winter operation where large temperature differentials are expected. Multiple cells designed to permit both series and parallel operation are recommended.
3. Express air requirements in terms of volume of air per pound of five-day BOD added or per pound of BOD removed, thus recognizing the effect of the strength of the waste on the air requirement.
4. Provisions for short term (24 hours or less) settling sections prior to the final effluent discharge to reduce solids carry-over due to agitation by the aeration device used.
5. Provide for proper location of inlets, outlets, aeration equipment, baffles, etc. to minimize hydraulic short circuiting and thus provide utilization of the full volume of the aerated lagoon for treatment.

It is obvious that equipment manufacturers and engineers must become involved and work with the regulatory agencies to produce aerated lagoon design criteria which will continue to stimulate the development of this economical waste water treatment process. It is essential that this economical technique of waste water treatment be adapted and improved upon to keep pace with an ever increasing pollution problem.

AERATED LAGOONS AT HIGH ELEVATIONS

by

Bill F. Pearson

There are several hundred towns, small communities, ski areas, campgrounds, and resorts located at an elevation greater than 5000 feet above sea level in the State of Colorado. This paper and design analysis of an aerated lagoon are based on experiences and observations in the Rocky Mountains of Colorado; however, there are thousands of similar communities and areas throughout the United States and other nations of the world. As the affluence of the world's population increases and transportation becomes more advanced, these areas are being used more extensively and wastewater disposal is becoming a more severe problem at the higher elevations. In addition, water quality standards are becoming more strict and the need for improved waste treatment facilities is increasing. Much of the land at high elevation is the source of the fresh water supplies for large segments of the world's population and there is a need to start maintaining the quality of these water supplies commencing at their point of origin.

The principal contributors of wastewater at most of the locations in the higher elevations are the small population centers. These small towns and communities doe not enjoy a large revenue or income with which to construct and maintain high quality wastewater treatment facilities. The typical maintenance man for such a community usually has additional responsibilities for maintaining roads, driving a school bus, operating the water system and, perhaps, acting as Deputy Sheriff; therefore, very little time is customarily allocated to the maintenance of the sewage treatment plant. Experience has shown that a stabilization lagoon treatment facility offers many advantages to such communities by lowering the initial costs, reducing the maintenance needs, and increasing the reliability of effluent quality$^{(1)}$. For the areas with seasonal population variations, the stabilization lagoon offers the additional advantage of being able to absorb heavy shock loadings between long periods of minimal loading. Ski areas, forest campgrounds and mountain resort towns are particularly characterized by shock loadings. In spite of the obvious advantages of a stabilization lagoon, experience with stabilization lagoons at higher elevations has not always been successful. A review of the desirable environmental conditions for the operation of a typical stabilization lagoon, as compared with the following environmental conditions experienced at many high elevation locations, will reveal that the operating conditions at high elevations are not always ideal:

1) Long winter months with deep snow and thick ice cover cause stabilization lagoons to be anaerobic for much of the year. In many instances, the lagoons cannot recuperate during the brief spring and summer period, thereby causing undesirable odors and poor-quality effluent.
2) Temperatures are usually much lower at high elevations, particularly in the mountains, and the biological activity is considerably reduced.
3) The atmospheric pressure diminishes as the elevation increases. The oxygen solubility is reduced and the recuperative ability of the pond is lessened.
4) There are other influencing factors, such as more difficult accessibility for maintenance, smaller sites for selection, freezing of influent and effluent lines, and reduced stream flows for dilution during the winter and fall seasons.

Aerated Lagoons

The introduction of aeration to a waste stabilization lagoon can eliminate many of the possible disadvantages and make the stabilization lagoon a more reliable waste treatment facility for the higher elevation. An aerated lagoon can be designed to serve an increased loading; therefore, the lagoon can be decreased considerably in size and less land is required. The reduction in size usually permits more flexibility in site selection and, in many cases, will permit the use of a site which could not be used for a conventional stabilization lagoon. Site selection always seems to be a major problem at higher elevations in the mountains because of adverse topography. In many cases, costly sewer mains and lift stations are eliminated by the additional flexibility in site selection.

The aerated lagoon maintains aerobic conditions at all times and eliminates the occasional odors experienced by conventional stabilization lagoons, particularly in early spring. The mechanically-introduced air eliminates the dependence on the wind for circulation and the sun for photosynthetic reaction to provide oxygen. The dissolved oxygen is maintained uniformly throughout the lagoon, and the odorous material is apparently oxidized and eliminated before it reaches the open air to become a nuisance$^{(2)}$. This factor takes the aerated lagoon out of the "bad neighbor" category and permits a lagoon site location to be more suitably located near the population centers.

Aerated lagoons are receiving recognition as a reliable means of waste treatment and efforts have been made to sophisticate aerated lagoon design. The basic design criteria seem to be relatively uniform among various authros. Increased experience and more studious monitoring of aerated lagoons under various loading conditions and environmental conditions are hopefully producing enough data to provide more specific design factors.

As with any treatment facility, an aerated lagoon can be made more functional and reliable by the inclusion of certain design features by the Engineer. Although a lagoon seems to be less susceptible to design errors than a mechanical plant, an unresearched design can create an aesthetic nuisance, a public health hazard, or a maintenance-demanding facility. Mechanical plants and aerated lagoons at high elevations seem to be particularly susceptible to design error because of the adverse environmental conditions.

Aeration Equipment

The heart of an aerated lagoon is the aeration system. The aeration system is particularly important at higher elevations because of the adverse environmental conditions. Many commercial suppliers are available to provide aeration equipment; however, the Engineer should, as on all projects, evaluate each individual design to specify a type of aeration equipment most suitable for the particular project.

Initially, a decision must be made as to whether a mechanical surface aerator or a diffused air system will be used. The available aeration equipment must be carefully studied to insure its workability in the field under all environmental conditions. Most suppliers have an adequate number of installations available for inspection purposes and the Engineer should prevail upon himself to examine such installations and evaluate their past operating experience.

(a) *Mechanical Surface Aerators*—There are cases on record of mechanical surface aerators which have functioned adequately under severe winter conditions over periods of several years. The ice on the surface of a lagoon tends to encroach upon a surface aerator, but a well-designed mechanical surface aerator will maintain an open space in the lagoon as a result of its vigorous mixing of the lagoon liquid. However, some mechanical surface aerators are apparently not as functional in extremely cold weather operations. Splashing liquids tend to freeze and accumulate on some equipment, and the organic loading of the lagoon becomes more important since clean water is more likely to freeze than a concentrated wastewater.

Surface aerator units are ordinarily not continuously operated and the equipment is highly susceptible to a total freeze-up during a prolonged shutdown period. The time of shutdown in the routine operation of a mechanical surface aerator must be evaluated in relation to the organic loading and specific site location. A maximum shutdown of only 15 minutes per hour may be permitted under some extreme weather conditions. Some suppliers and engineers provide summer and winter timer clocks for the operation of the mechanical surface aerators rather than leave the adjustment of a timer control to the maintenance personnel.

The surface of an aerated lagoon can become totally frozen during a prolonged shutdown of the aeration equipment; therefore, the Design Engineer must consider the possibility of potential power outages and equipment malfunctions. Experience has shown that power outages can be expected to occur at least several times per year in rural or isolated areas. Most power outages are brief, but extended outages do occur. If the situation is critical and the power service is evaluated as unreliable, a standby generator can be provided for immediate automatic use. An Engineer could find a well-designed aerated lagoon to be the victim of a downed power line during a blizzard, and such an accident could freeze the lagoon for the remainder of the winter. This could be a real problem if the aeration equipment is susceptible to damage by surface freezing in the lagoon.

(b) *Diffused Air Equipment*—Diffused air equipment does not appear to be as susceptible to adverse weather conditions since an underwater diffused air system can function with the surface of the lagoon completely frozen. The compressor equipment should be housed to protect it against the elements and to provide for ease of maintenance. Air piping and all components of the diffused air system should be installed on the floor of the lagoon or submerged sufficiently deep beneath the water level to prevent ice damage during a lagoon freeze-up. The greatest danger seems to be from breakage of the pipelines on the edge of the lagoon by movement of the ice on the surface. There is little danger of internal freezing causing breakage in the compressed air pipelines because of the natural heat of compression. The water in a lagoon can easily freeze to a depth of 3 feet during a prolonged shutdown and provisions must be made to protect piping, submerged cones or other aeration appurtenances from this type of damage.

The diffuser heads or the perforated pipe for diffused air equipment must be installed with adequate provisions for maintenance against plugged orifices or malfunctions. The diffuser heads should be designed for ease of removal for repair or cleaning. For perforated pipe, it is advisable to install acid-rinsing equipment for routine cleansing of the orifices. This type of maintenance and repair work must be able to be performed without draining the lagoon and should be routinely scheduled for the fall season to avoid the more difficult winter repairs.

Temperature

The temperature of the aerated lagoon is obviously a major factor in the BOD reduction. The bacteria in stabilization lagoons are capable of growth between 0° and 42°C, but optimum growth occurs between 22° and 25°C(3). Summer temperatures permit good biological action, but winter temperature in most aerated lagoons at higher elevations are in the range of 0.5° to 15°C and biological activity is often at a minimum. The depth of the lagoon should be designed to retain the inherent heat of the liquid waste. Heat is lost by convection, radiation and evaporation. A deep lagoon with a small surface area will lose less heat than a shallow lagoon with a large surface.

The temperature of the liquid in the lagoon depends upon the temperature of the incoming waste, the ambient temperature, the organic loading, and the heat contribution, if any, by the aeration equipment. Lagoons for new sewage disposal systems do not usually have an adequate organic loading to significantly affect the temperature. A heat contribution is apparently made by compressed air equipment in the nature of heated air and warmed piping, but the author has no operational data to substantiate this observation.

During summer there is an increase in the lagoon temperatures as a result of solar radiation, but this contribution is negligible during the winter when ice cover is on the lagoon. At the present time, the author is not aware of any efforts to increase lagoon temperature by an external source to accelerate the biological activity during the winter. However, this is being tried in other forms of waste treatment and should be investigated for aerated lagoons, particularly those at very high elevations where temperatures are extremely low. A review of the design analysis revel that a 5° rise in lagoon temperature from 0.5° to 5.5°C will considerably increase the K_t factor and increase the efficiency of the lagoon.

The design engineering related to the heating of an aerated lagoon can be accomplished, but the economics must be thoroughly evaluated to determine its practicality. The calculation can get difficult since it is not strictly a matter of calculating the BTU's of heat necessary to increase a given quantity of liquid to a certain level. In this particular application, as the heat is applied it may have the effect of melting the surface ice and increasing the surface exposure to the cold air, thereby radically increasing the rate of heat loss.

A research facility could investigate to determine if an optimum economical rate for the application of heat does exist for an aerated lagoon, but it is obviously not within the scope of a typical aerated lagoon design problem. There may be an optimum temperature to which the biological activity can be increased in an aerated lagoon with a minimum effect on the insulating capability of the ice and snow cover. The increase in lagoon efficiency would have to be sufficient to justify the additional cost. It is even possible that certain strip covers or shields could be researched to determine an economically feasible means of retaining the heat within an aerated lagoon.

Liquid Mixing and Oxygen Dispersion

The temperature and the reduced atmospheric pressure are significant factors in the calculation of the biological oxygen required; however, the design of the aeration system is usually governed by the requirements for liquid mixing and oxygen dispersion. The calculations of the required aerator horsepower to insure liquid mixing and oxygen dispersion are usually greater than that required to provided for biological oxygen.

The oxygen level in the lagoon must be greater than 1.0 part per million in a uniformly-mixed liquid or the oxygen up-take rate in the aerated lagoon will be oxygen-concentration dependent(4). The aeration equipment should be specified on a performance basis and the specifications should permit the Engineer to evaluate the treatment system at less than design loadings and at various temperatures(5). The period of equipment evaluation for acceptance inevitably occurs during the initial stages of the aerated lagoon when loading conditions are far below the design loading.

Lagoon Configuration and Design Details

Aerated lagoons should consist of two or more cells for increased flexibility if feasible. Series operation should produce a higher degree of removal for solids, BOD, and coliform for an aerated lagoon as for a conventional stabilization pond(6). The Engineer should take advantage of the increased flexibility of two cells by constructing the inlet lines, connecting lines, and discharge piping to permit either cell to be bypassed or drained for maintenance or repair purposes. One apparent advantage of parallel operation of two cells seems to be distribution of the solids over the entire lagoon area(7). Most authors have not found solids accumulation to be a significant factor in lagoon operations for domestic wastes; consequently, series operation is favored. It is important to provide a quiescent area to prevent solids from being carried out of the pond in the effluent discharge. The quiescent area can usually be provided by the spacing of the aerators or a reduction in the aeration near the discharge.

An aerated lagoon at a high elevation should have a submerged inlet, a protected outlet, and adequate freeboard, as well as other design details which are common to lagoon design for cold weather climates. Many aerated lagoons at high elevations have a problem with effluent discharge during the winter. A submerged discharge in the receiving water is desirable, but not always available. Some aerated lagoons at very high elevations may have to store the liquid until spring or discharge the effluent in a place where the frozen effluent will not be a nuisance or public health hazard.

Disinfection of Effluent

Aerated lagoons should have provisions for chlorination of the effluent. The chlorination should be controlled by an automatic feed rate or, at the very least, a manually-adjustable timer clock to prevent gross over-dosages of chlorine.

Design Analysis

A design analysis should be prepared for each aerated lagoon design and submitted for review and approval by the appropriate governmental agencies and the client. The design analysis should contain design criteria, design calculations, the proposed facilities and other pertinent information. The design analysis included as Appendix 1 of this report is a typical example of the format used by the author and his associates for aerated lagoon designs in the State of Colorado at elevations as high as 11,600 feet above sea level. Operating information is being collected from these lagoons in an effort to improve the design; however, limited data is available from field operations at this time.

Conclusions

1. The addition of aeration to wastewater lagoons will increase their reliability for operation at high elevations.
2. Low temperatures and severe winter weather are the most significant environmental factors affecting aerated lagoons at high elevations.
3. The aeration system for a lagoon at a high elevation must be selected and designed to avoid equipment damage by freezing during winter operations.
4. Mechanical surface aerators and diffused air equipment have both been used successfully at high elevations. Surface aerators are more susceptible than submerged aerators to damage or stoppage as a result of a total freeze-up of the lagoon surface during extremely severe winter weather.
5. The design of the aeration system capacity is usually governed by requirements for liquid mixing and oxygen dispersion.
6. A two-cell lagoon with series operation and a quiescent zone near the effluent discharge is favored by the author for a higher degree of removal for solids, BOD, and coliform.

Appendix 1

Design Analysis
For an Aerated Lagoon Facility
Nederland, Colorado

1. Introduction

The scope of this project includes the construction of sewage collection and treatment facilities to serve the business area and the outlying residential areas of the Town of Nederland, Colorado. At the present time, no central sewage collection or treatment system exists. The town relies on individual septic tanks with leaching fields or privies for sewage disposal.

Location

Nederland, Colorado, is located in the valley of Middle Boulder Creek at its juncture with North Beaver Creek, approximately 17 miles west of Boulder, Colorado A mountain community in the front range of the Rockies, it is situated on the alluvial plains of the creeks passing through the town. Development of the higher land, adjacent to the town proper, is now in progress. Barker Reservoir, owned by the Public Service Company of Colorado, lies immediately east of the town.

Water Supply

The Town of Nederland has a central water supply system which includes a water treatment facility, storage tanks and a distribution system through the major portion of the town extending into adjoining subdivisions. The source of the water supply is Middle Boulder Creek at a point upstream from Nederland. The water is filtered through a slow sand filter and chlorinated prior to usage. The distribution system was reconstructed and upgraded in 1965.

Population

The present permanent population of Nederland is approximately 600. The town's population increases to approximately 800 during the summer season when the summer cabins are occupied. Many people will not be served initially because Schedule 1 of the proposed sewer system will not extend sewers to every home within the city. In addition, many of the homes presently have individual septic tank systems. The initial equivalent population to be served by the sewer system will be approximately 500. The projected growth of the population to be served by the sewer system is shown in Table 1. The design population for the sewage treatment facilities is 888.

Projected Growth of Population to be Served

Year	*Increase per Year*
First year	44
Second year	40
Third year	37
Fourth year	33
Fifth year	30
Sixth year	26
Seventh year and after	22

TABLE 1

2. *Proposed Sewage Treatment Facilities*

The sewage treatment plant for this initial construction project will consist of an aerated stabilization pond and an aerated polishing pond. Facilities for post-chlorination of the polished pond effluent are being installed to ensure high quality effluent. The plant effluent will be discharged into Middle Boulder Creek as the stream enters Barker Reservoir. Barker Reservoir is an oligotrophic lake with a capacity in excess of 12,000 acre-feet. Middle Boulder Creek is a continually flowing stream which has an average daily flow in excess of 36 million gallons per day based on records since 1908.

The plant has been designed for future unit expansion according to the population growth of the entire service area. Land area and hydraulic head have been included in the plant design to provide for the future addition of a mechanical sewage treatment plant if needed. The future mechanical plant can provide primary treatment and nutrient removal if required at that time. The aerated stabilization pond and polishing pond will then be used for high quality secondary treatment and as a polishing pond.

Aerated Stabilization Ponds

The two aerated ponds will be adequately provided with aeration to maintain a minimum dissolved oxygen value of 2 ppm at design loading. The equipment provided will be designed to provide 3 ppm dissolved oxygen in the ponds at the loading contributed by the population being served of approximately 500 and a DO of 2 ppm for the design population of 888.

The treatment facility is designed to produce a BOD reduction greater than 90% for the design population of 888.

The aeration pond shall consist of 2 cells operated in series. The liquid depth will be 10 feet and 3 feet will be allowed for freeboard. Interior pond corners will be rounded.

Evaluation of the soil conditions on the site indicated 2.5:1 dike slopes were permissible. The 2.5:1 slope will have good stability and will minimize the algae and week growth which is often found in the shallow edges of stabilization ponds with shallow side slopes. The lack of weeks and stagnant water on the ponds' edges will also prevent the growth of mosquito larvae. The berm of the dike will have a minimum 8-foot width and will be used as an access road for maintenance vehicles.

Inlet pipes will be placed on concrete slabs slightly below pond grade. The pond discharge structure will be located at the opposite end of the lagoon from the inlet structure to prevent short circuiting. The discharge lines shall rest on a concrete apron and will draw liquid from varying levels beneath the pond surface.

Drainage lines shall be installed to permit the lowering of the water level or the complete drainage of each individual pond if necessary. The drainage outlet will be within a concrete box constructed on grade to prevent erosion of the pond bottom during drainage. Calcium hypochlorite will be stored in the chlorine house to be used for batch chlorination of the ponds in the event drainage becomes necessary.

The interconnecting manhole will have a flow-through weir device to minimize the effects of peak flows by permitting a 2-inch rise in the water level of Pond No. 1 and again in Pond No. 2 during a peak flow. The removable weir may be altered in design to permit any desired retention characteristic. A calibrated flume will be installed in the effluent sampling manhole.

The earthwork construction on the dikes will primarily be accomplished with material available on and near the site location. The pond will be completely lined with a 6-inch blanket of high quality clay to ensure minimum seepage and to maintain the design water depth of 10 feet. A valved water inlet pipe will be constructed to provide creek water into the sewer system near the lagoon for the purpose of adding water to the ponds as needed.

The ponds will be terraced and landscaped to present an esthetically desirable appearance appropriate to the mountain locale. It is planned that dikes will be seeded, and shrubbery and trees will be planted.

A stock-type fence will encompass the plant facility as needed. An access gate with locks will be provided.

Warning signs will be erected along the perimeter of the facility to designate the nature of the facility and advise against trespassing.

Disinfection

Facilities will be provided for post-chlorination of the polishing pond effluent. The chlorination will be by chlorine solution injection into a 2000-gallon chlorine contact chamber. The contact chamber will provide greater than 30 minutes contact period at average flow and a minimum of 15 minutes contact at twice the daily average flow. The interconnecting manholes are designed to reduce the influence of peak flows by permitting the pond elevation to rise approximately 0.15 feet.

The chlorination house is designed to operate without heating facilities. The water used in the ejector solution is recirculated from the chlorine contact chamber and the chlorination house will not contain any water. An exhaust fan, leakage kit, gas mask, safety chain and safety instructions are being provided in accordance with the State Health Department regulations.

Design Criteria - Sewage Treatment Plant

1. Design population of present project
   - Summer population 888
   - Winter population 665
2. Average daily per capita sewage flow . . . . .100 gallons
3. Average daily per capita BOD contribution 0.20 lb 5-day BOD
4. Average 5-day BOD . . . . .240 ppm
5. BOD reduction through plant . . . . . greater than 90%
6. Minimum dissolved oxygen in aerated ponds . . .2 ppm
7. Elevation of treatment plant . . 8,200 feet above MSL
8. Mean Air Temperatures (Winter . . . . .25°F
   - (Summer) . . . . .60°F
9. Mean temperature of sewage influent (Winter) . .45°F
   - (Summer) . . . . .55°F
10. Mean temperature of aerated ponds (Winter) . . . .34°F
    - (Summer) . . . . .58°F

Design Calculations - Aerated Lagoons

(A) Retention Time

Formulae for calculation of retention time and BOD rate removal coefficient

$$T = \frac{\frac{Lo}{Le} - 1}{K_t} \qquad \text{(Equation 1)}$$

where:

T = retention time in days

Lo = wastewater influent strength in ppm

Le = wastewater effluent strength in ppm

K_t = BOD rate removal coefficient at lagoon temperature (t)

$$K_t = K_{20^\circ C}\,\theta^{(t-20)} \qquad \text{(Equation 2)}$$

where:

$K_{20^\circ C}$ = 0.5 (value ranges from 0.3 to 1 for domestic sewage; 0.5 is often used as a conservative value$_{(9)}$)

θ = 1.035 (temperature correction factor for aerated lagoons; value ranges from 1.016 to 1.047$_{(4)}$; recent studies have shown that 1.035 is a good factor for continuous-flow conditions)

Calculation of retention time necessary for summer operation

For summer operation at 15°C

$K_t = 0.5(1.035)^{15-20} = 0.42$ Ref: Equation 2

to produce a pond effluent BOD of 30 ppm

$$T = \frac{\frac{240}{30} - 1}{0.42} \doteq 16.7 \text{ days} \quad \text{Ref: Equation 1}$$

Calculation of retention time necessary for winter operatio

$K_t = 0.5(1.035)^{2-20} = 0.27$ Ref: Equation 2

To produce a pond effluent BOD of 30 ppm

$$T = \frac{\frac{240}{30} - 1}{0.27} = 26 \text{ days} \quad \text{Ref: Equation 1}$$

The winter operation requires a longer retention time to produce the desired BOD reduction: therefore, the design retention time will be 26 days.

(B) Calculation of Summer Efficiency - Based on Design Retention Time

$$\text{Efficiency} = \frac{100\,KT}{KT + 1} \qquad \text{(Equation 3)}$$

Where:

K = .42 (Summer operation at 15°C as calculated above)

$$\text{Summer Efficiency} = \frac{100(.42)(26)}{.42(26) + 1} = 91.6\%$$

BOD of pond effluent is calculated to be 20 ppm

NOTE: The pond BOD effluent values calculated above will further reduced by approximately 2 ppm for each ppm of applied chlorine. A feed rate of 6 ppm is proposed; therefore, the BOD in the effluent should be reduced by an additional 10-12 ppm.

(C) *Required Pond Capacity*

Pond Capacity (c) = Average daily flow x design retention time

$$c = 665 \text{ people} \times 100 \frac{\text{gal}}{\text{day}} \times 26 \text{ days} = 1{,}729{,}000$$

gallons based on
winter population
and retention time

Use 2 ponds each with 865,000-gallon capacity

(D) *Biological Oxygen Requirements*

For maximum condition—summer operation
lbs of oxygen required per day = L x BOD x % BOD removal

Where:
L = 1.4 (a coefficient used for domestic sewage as the ratio of ultimate BOD to 5-day BOD)

$$\text{lbs of oxygen required per day} = 1.4(240 \text{ ppm} \times 88{,}800 \frac{\text{gal}}{\text{day}} \times 8.34 \frac{\text{lb}}{\text{gal}} \times .947$$

$$= 233 \text{ lb/day}$$

$$O_{2s} = \frac{O_{2a} C_s}{a(C_{sw} \times \frac{P_a}{P_s} - C_L)\ 1.02^{t-20}}$$

Where:

O_{2s} = Oxygen required at standard conditions - lb/day

C_s = Dissolved oxygen saturation value at 20°C = 9.17

O_{2a} = Oxygen required under actual conditions - lb/day

a = Oxygen transfer ratio of waste to water, normally 0.9 for domestic waste

C_L = Oxygen concentration to be maintained in lagoon operating temperature - ppm = 10.15 at 15°C

C_{sw} = Dissolved oxygen saturation value at lagoon operating temperature - ppm = 10.15 at 15°C

P_a = Barometric pressure at lagoon site (in.Hg) = 22.50

P_s = Barometric pressure at standard condition (29.92) in. Hg.

t = lagoon contents temperature (°C)=15°C during summer

$$O_{2s} = \frac{233(9.17)}{0.9(10.15 \times \frac{22.50}{29.92} - 2)\ 1.02^{15-20}}$$

$$= 463 \text{ lbs/day} - 19.3 \text{ lb/hr}$$

Ref: Equation 4

(E) *Aeration Horsepower Required*

Assuming 3.2 lb O_2 Hp-Hr

$$HP = \frac{19.3 \text{ lb/hr}}{3.2 \text{ lb } O_2/\text{Hp-Hr}} = 6 \text{ HP}$$

or an 8 HP aerator operating 75% of the time

NOTE: The above calcuiated value is a minimum value. The aeration equipment installed will be able to furnish aeration or aerator horsepower to ensure adequate dissolved oxygen, mixing, and dissovled oxygen dispersion to provide for the total oxygen requirements of 19.3 lb/hr.

(H) *Required Aerator Horsepower for Liquid Mixing and Oxygen Dispersion*

Assuming 8 HP per million gallons lagoon capacity
Pond Capacity 1.73 million gallons
HP = 8HP x 1.73 = 13.8 HP

NOTE: Plant design is for 2 rectangular ponds. Therefore, approximately 4 aerators providing 5HP each operating 69% of the time would be required for mixing and oxygen dispersion, if mechanical surface aerators are used. The selected aeration equipment will be sized in accordance with the manufacturer's recommendations.

(I) *Chlorination*

Volume of chlorine contact chamber = V =

$$V = \frac{888 \text{ people} \times 100 \frac{\text{gal}}{\text{day}}}{60(\frac{\text{min}}{\text{hr}}) \quad 24 (\frac{\text{hr}}{\text{day}})} \times 2(\text{peak flow factor})$$

$$\times 15 \text{ minutes (contact time)}$$

V = 1850 gallons
Use a 2000-gallon pre-cast concrete tank
Chemical rate of feed = 6ppm at twice design flow

$$\text{Chemical feed} = 888 \text{ people} \times 100 \frac{\text{gal}}{\text{day}} \times 8.34 \frac{\text{lb}}{\text{gal}}$$

$$\times 6\text{ppm} \times 2(\text{peak flow factor})$$

$$= 8.9 \text{ lb/day maximum chemical feed}$$

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TERTIARY TREATMENT BY AERATED LAGOON

BY

V. N. Wahbeh and L. W. Weller

INTRODUCTION

With the advent of stringent water quality criteria on all interstate and many intrastate streams, the necessity to improve the quality of secondary treatment plant effluent is imminent. Municipalities using trickling filters for secondary treatment of their wastes, and in some cases those using a conventional activated sludge process, will have to take a second look at their treatment efficiencies.

Trickling filter operation is greatly dependent on two parameters, the temperature of the liquid and the recirculation rate. A drop in temperature means a drop in the efficiency of treatment by the trickling filter. For instance, if the BOD removal efficiency of the trickling filter is about 80 per cent at 24°C, this efficiency will drop to 70 per cent at 20°C. High rate trickling filters are also dependent on recirculation for good BOD removal efficiencies, and recirculation rates of 2:1 to 4:1 are not umcommon. Theoretically a trickling filter that is 6 feet deep and has a hydraulic loading of 15 million gallons per acre per day has a 90 per cent BOD removal efficiency at a recirculation rate of 4:1. Without recirculation the same filter will have a BOD removal efficiency of only 70 per cent. The operating costs for high recirculation rates to attain good removal efficiencies are very high, and even then, drops in temperature might cause the expected efficiencies to drop below normal. In most existing facilities, increasing recirculation is impossible without very extensive piping revisions and increasing the capacity of the pumping facilities.

To improve the quality of the effluent from existing facilities, third stage of treatment will have to be added. Although third stage treatment, or tertiary treatment as it is known today, is normally associated with phosphate removal and the many sophisticated processes that can turn sewage plant effluent into drinking water, in this presentation the terms "tertiary treatment" or "third stage treatment" will only deal with upgrading the quality of the secondary treatment plant effluent as far as BOD and suspended solids are concerned, and generally, making the effluent less objectionable for discharge into the receiving stream. In this presentation an experimental process will be discussed that was used at the Johnson County, Kansas, Indian Creek sewage treatment plant to treat trickling filter effluent by an aerobic lagoon to improve its quality before discharge to the receiving stream. Performance data on this lagoon, both as an oxidation pond and later as an aerated lagoon, will be presented and discussed.

BACKGROUND INFORMATION

The lagoon, which was one of two originally built in 1956 to serve a population of 3000, was kept in service when a permanent treatment plant was constructed. The purpose for keeping this lagoon in operation was twofold: (1) to treat flows reaching the treatment plant that are in excess of the plant hydraulic capacity of 250 gallons per capita per day, and (2) to provide additional treatment for the effluent of the trickling filter serving a population of 10,000.(1)

Since it was believed the lagoon did an effective job of treating the trickling filter effluent as far as reducing the BOD and suspended solids, it was decided, when the plant was expanded to serve an increased population of 30,000, to keep the lagoon in service to provide third stage treatment for the plant effluent and to obtain some operational data on the performance of this type of lagoon.

In 1965 the plant capacity was increased again to serve a population of 70,000. At this design it was felt that the average loading on the lagoon estimated at 450 pounds of BOD per acre per day would be too high for efficient operation of the lagoon. It was decided to rebuild the lagoon to make it suitable for equipping with aeration equipment to maintain the lagoon contents aerobic, and thus provide adequate treatment to the trickling filter effluent. The revisions to the lagoon included raising the berm elevation, installing a new effluent structure, scraping and deepening the lagoon bottom and providing aeration equipment which was of the "Aqua-Air Oxidation System". This type of aeration consists of a grid of weighted tubes laid on the lagoon bottom. Small check valves in the piping form air loutlets. Air is provided by three 600 cfm blowers.

DESCRIPTION OF TREATMENT PLANT.

The Indian Creek treatment plant Figure 1 is a typical trickling filter plant. It consists of mechanically cleaned screens, a pumping station, grit chamber, primary sedimentation tanks, high rate trickling filters, and final sedimentation tanks followed by the lagoon. The plant is designed so that during wet weather periods flows in excess of plant capacity could bypass the plant and discharge directly to the lagoon. This procedure has not been followed in recent years. In the latest expansion, chlorination facilities were added following final sedimentation, just ahead of the lagoon.

The plant is located at the confluence of Indian Creek and Tomahawk Creek in Kansas. The lagoon effluent is discharged into Indian Creek at a point from which the creek flows for about one mile in Kansas and then 3-½ miles in Kansas City, Missouri, through very expensive suburban areas before finally discharging to the Blue River.

RESULTS AND DISCUSSION

Oxidation Ponds. In 1963 Loehr and Stephenson(2, 3) from the University of Kansas made a study of the performance of this lagoon before the aeration system was installed. The treatment plant was serving a population of 20,000 and had a flow of 1.5 to 2.5 mgd, a surface area of 7 acrrs, a detention time of 2.4 to 4 days, and a BOD loading of 17 to 51 pounds of BOD^5 per acre per day as shown in Table 1.

Table 1

OXIDATION POND

Summary of Design Parameters

Flow into Lagoon	1.5 - 2.5 mgd
Detention Time	2.4 - 4.0 days
Surface Area	7 acre
BOD^5 Loading	17 - 51 lbs BOD^5/acre/day

The lagoon was 2'-6" deep and had side slopes of 3:1. Loehr's and Stephenson's (2, 3) study was quite extensive. Samples were taken every hour of both the lagoon influent and effluent and tests were run on both the hourly samples and the 24 hour composits. Tests included BOD, COD, suspended soilds, all forms of nitrogen, phosphate, pH, and dissolved oxygen. For the purposes of this presentation, only the BOD and suspended solids of the influent and effluent and the dissolved oxygen of the lagoon contents will be presented and discussed. Since their data has been previously published, no attempt has been made to present all their data, but what is presented is typical.

Dissolved Oxygen and pH in Lagoon. Oxygen concentrations as determined by Loehr and Stephenson(2, 3) indicate that the pond was supersaturated during the daylight hours in the summertime, while dropping to about 6 mg/l during the night. In all cases the oxygen concentration was higher than that of the influent of the lagoon. In the wintertime, however, this was not true and oxygen concentrations in the effluent were lower than in the influent, 2 to 3 mg/l in the effluent and 4 to 5 mg/l in the influent to the lagoon. pH values in the lagoon averaged above 8.6 furing the summer months and about 7 to 7.5 during the winter months. The dissolved oxygen and pH data indicate the high rate of algal activity in the summertime, and the added organic load on the lagoon and decreased algal activity during the winter months.

BOD and Suspended Solids: Figure 2 shows the BOD in the influent to the lagoon during one representative day in the summertime and one representative day in the wintertime. This figure also indicates the effect of temperature on the trickling filter efficiency.

In the wintertime the BOD load into the lagoon was essentially twice the load to the lagoon in the summertime. Figure 3 shows the suspended solids in the influent of the lagoon during the same two days. Both of these figures show hourly variations of both the BOD and the suspended solids as affected by the influent to the treatment plant. Figure 4 shows the BOD of the effluent from the lagoon while Figure 5 shows the suspended solids in the effluent of the lagoon during the same two days, July 10-11 and December 1-2, 1963.

No significant BOD or suspended solids removal efficiencies are apparent from the data collected during the summer months. This was due to the algae concentrations in the lagoon and the effluent causing the high BOD values. This BOD was a fairly stable type since it was in the form of algal cells and did not create any objectionable conditions immediately downstream. The algae, though, will eventually die, release organic and inorganic nutrients, and create an oxygen demand and associated obnoxious conditions.

Fairly good BOD and suspended solids removal efficiencies occurred in the winter months, sometimes as high as 60 per cent. This removal is misleading, however, since it is due to the settling of solids to the bottom of the lagoon. Part of the BOD in these settled solids would eventually be released in the warmer months, creating an additional load on the lagoon. Sudden and periodic algal losses were reported by Raschke(5); at an experimental tertiary pond at Ames, Iowa, during these periods the BOD in the effluent of the lagoon was substantially higher than that in the influent.

BOD reductions of 20 to 60 per cent were obtained from the operation of this lagoon, with an overall BOD reduction of about 40 per cent for the sampling days.

From the data presented by Loehr and Stephenson (2, 3) Weiss (4), and Raschke (5) it can be concluded that the treatment provided by an oxidation pond to the trickling filter effluent is questionable as far as BOD and suspended solids are concerned, since the oxidation pond merely changes the form of the BOD. This is also true as far as inorganic nutrients are concerned. The algae have to be removed by some means from the effluent of the lagoon; then and only then will good BOD, suspended solids and inorganic nutrients be obtained.

The oxidation pond has some advantages however as a holding tank for leveling out the variations in the strength and quality of the effluent. Studies by Weiss (4) in North Carolina indicated that the tertiary oxidation pond was very effective in reducing coliform numbers. Efficiencies of 95 per cent were obtained without exception.

Aerated Lagoon. As mentioned earlier, due to the Indian Creek treatment plant expansion and the expected increase in the load to the lagoon, the oxidation pond was rebuilt and equipped with aeration equipment.

After the installation of the aeration equipment, performance tests were run on the lagoon. These tests included a two week sampling program to determine the

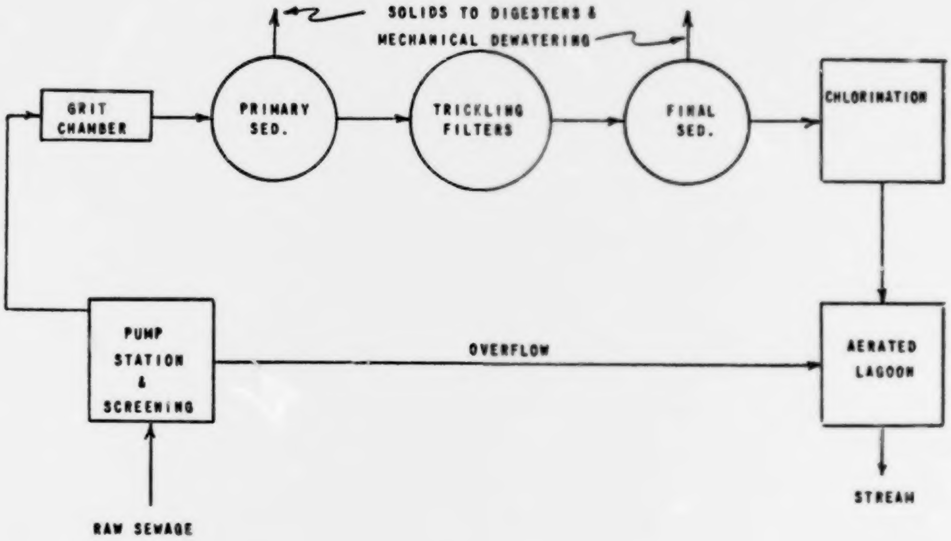

SOLIDS TO DIGESTERS &
MECHANICAL DEWATERING
GRIT CHAMBER
PRIMARY SED.
TRICKLING FILTERS
FINAL SED.
CHLORINATION
PUMP STATION & SCREENING
OVERFLOW
AERATED LAGOON
RAW SEWAGE
STREAM

Figure 1. Schematic Indian Creek Treatment Plant

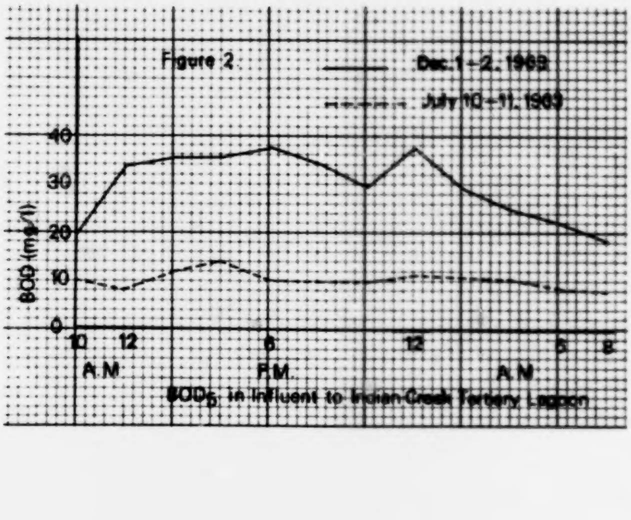
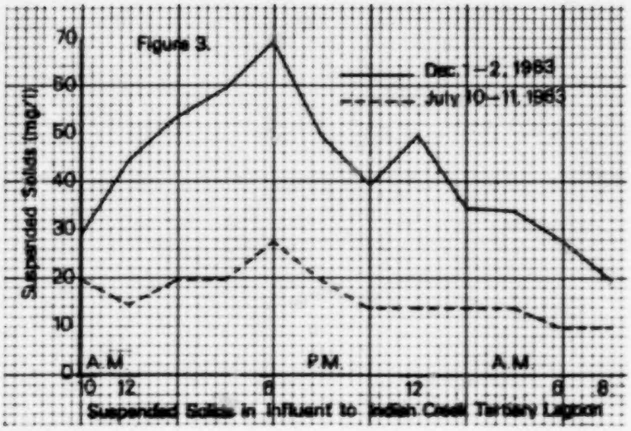


Figure 2

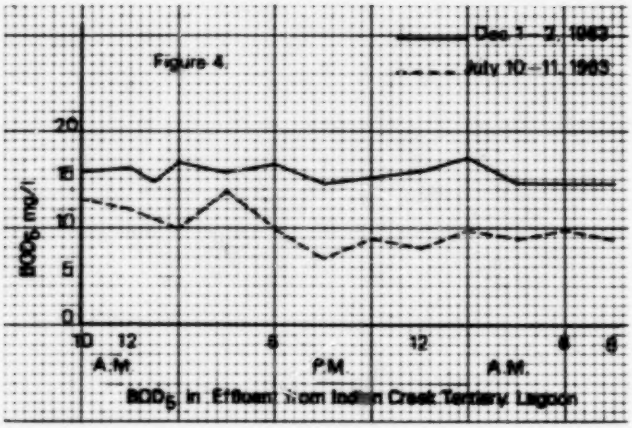
BOD_5 in Influent to Indian Creek Tertiary Lagoon

Figure 3.



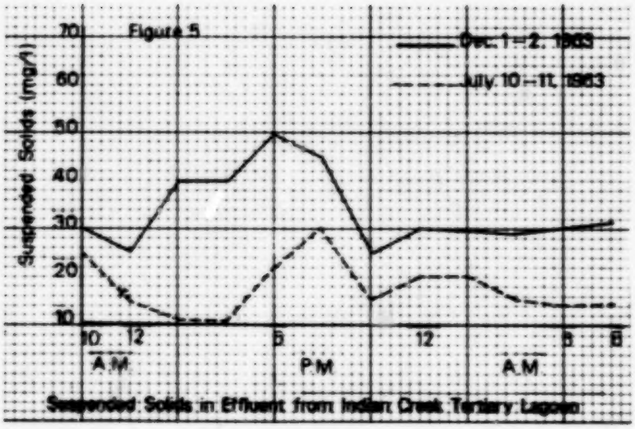
Suspended Solids in Influent to Indian Creek Tertiary Lagoon

Figure 4.



BOD_5 in Effluent from Indian Creek Tertiary Lagoon

Figure 5



Suspended Solids in Effluent from Indian Creek Tertiary Lagoon load on the lagoon and the efficiency of treatment. Samples were collected by an automatic sampler each hour and composited at the end of 24 hours. Other performance tests included measuring the dissolved oxygen concentrations throughout the lagoon, and the oxygen transfer capability of the aeration equipment.

Table 2 shows some of the operational parameters of the aerated lagoon. At the time when performance tests were made during November of 1968 the flow averaged between 4 and 7 mgd resulting in a lagoon detention time of 1.4 to 2.4 days. The BOD loading on the lagoon at the time of testing ranged between 60 to 178 pounds per acre per day with an average load of 100 pounds of BOD per acre per day. Lagoon side slopes were 4:1 and lagoon average operating depth was 5 feet, providing a surface area of 6.2 acres.

Table 2

AERATED LAGOON

Summary of Design Parameters

Flow into Lagoon	4.0 - 7.0 mgd
Detention Time	1.4 - 2.4 days
Surface Area	6.2 acres @ 5 ft. depth
BOD^5 Loading	60 - 178 lb/acre/day
	Avg. 100 lb/acre/day

Figure 6 shows the BOD of the lagoon influent and effluent for November 4 through 14. Figure 7 shows the suspended solids in both the influent and the effluent of the lagoon for the same period. Two of the compressors provided with the aeration equipment were in operation for the duration of the testing period. Results consistently show better than 50 per cent BOD and suspended solids removal except for the last day of sampling when, due to heavy rains, the manually cleaned screens ahead of the grit chambere became plugged and the raw sewage overflowed into the lagoon. Overall BOD reduction for the two week period was 60 per cent.

Figure 8 shows the monthly average BOD^5 and suspended solids in the lagoon effluent for 1969. The important thing to notice is the uniform quality of the effluent throughout the year.

The operation of the aerated lagoon indicates that due to the short detention times in the lagoon the opportunity for algal cell synthesis is reduced. The lagoon effluent has very little algae which is substantiated by the effluent BOD. It is suspected however, that the major part of the effluent BOD leaving the pond is due to algae. This is especially true during periods of prolonged warm, sunny and dry weather conditions. At these weather conditions, detention periods in the lagoon are longer than the indicated average, allowing more time for algal cell generation. The pH during these periods does not exceed 7.5, which indicates that algal activity is not quite as high as it would be in an oxidation pond with shallow water depths and without aeration.

Temperatures and Dissolved Oxygen Concentrations in the Lagoon. With two compressors in operation, the dissolved oxgyen concentration in the lagoon varied from a low of 7.4 mg/l at the influent end of the lagoon to 7.9 mg/l at the effluent end of thy lagoon. Temperature readings indicated that the temperature was uniform throughout the lagoon. There was no variation in temperature or dissolved oxygen with depth. With one compressor in operation, the dissolved oxygen concentration in the lagoon varied from a low of 6.2 mg/1 at the influent end, 6.4 mg/l at the effluent end, and 6.6 mg/l at the middle of the lagoon. Temperature readings varied from 14°C readings at the influent end to 12°C at the effluent end. There was no variation in temperature or dissolved oxygen concentration with depth.

DESIGN CRITERIA

It is interesting to note the actual BOD loading on the lagoon, both when it was operated as an oxidation pond, and as an aerated lagoon. Design Criteria do not differentiate between lagoons treating raw sewage and lagoons treating secondary plant effluents. The BOD in raw sewage is in an unoxidized form with very high oxygen demands requiring immediate satisfaction. On the other hand, the major part of the BOD in secondary plant effluent is in an oxidized form with a very low oxygen demand. Consequently, higher loadings are possible on oxidation ponds treating secondary plant effluents.

Design criteria are not available for aerated lagoons treating secondary effluents. The approach that was taken in the design of the Johnson County Indian Creek aerated lagoon was (1) determining the maximum amount of BOD that would reach the lagoon, (2) determining the oxygen requirements to satisfy this BOD, (3) specifying the amount of oxygen that the aeration equipment had to transfer, and (4) testing the performance of the aeration equipment after its installation to determine whether it had the capacity to transfer the specified amount of oxygen under operational conditions. Tests made included measuring the BOD load to the lagoons, efficiency of treatment, and oxygen uptake tests of the influent and of the lagoon contents. Oxygen uptakes were used in the equation presented in Figure 9 to determine the $K^{\alpha}a$ value of the lagoon influent. Oxygen uptakes were determined by using a direct reading dissolved oxygen probe and measuring the oxygen depletion in a stirred, but not aerated sample. Figure 10 shows a typical plot of oxygen depletion versus time, the slope of the plot being the oxygen uptake of the sample. The resulting oxygen uptakes ranged from 0.5 to 0.7 mg/l/hr which, when plugged into the oxygen transfer equation (Figure 11), yielded the transfer capabilities of the aeration equipment. Table 3 shows results of oxygen transfer in 16/day with two compressors in operation. The specified oxygen transfer capability was 4000 pounds with all three compressors in operation.

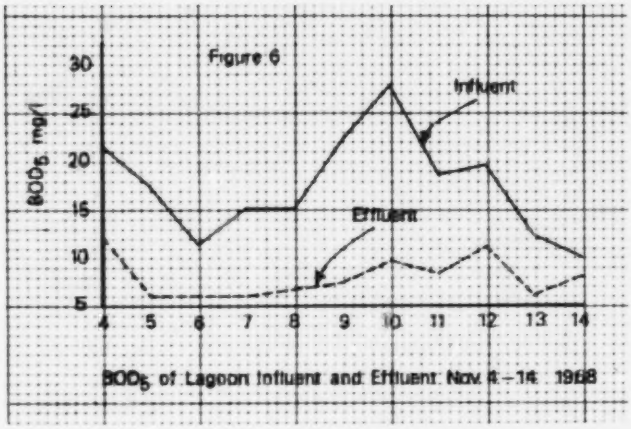
Table 3

RESULTS OF OXYGEN TRANSFER DETERMINATIONS

Nov. 19, 1968	3750 lbs/day
Nov. 20, 1968	3500 lbs/day
Nov. 22, 1968	2800 lbs/day
	Average 3350 lbs/day

The aerated lagoon is provided with 4:1 side slopes and has a water depth of 5 feet. Solids deposition has been occurring on the slopes where there is no aeration and mixing. During maintenance of lagoon aeration equipment, the water level is brought down, the algae growth on the deposited solids intensifies and buoys clumps of these solids

Figure 6



BOD$_5$ of Lagoon Influent and Effluent Nov. 4 - 14 1968

Figure 9

Oxygen Uptake Equation

$$\frac{do}{dt} = K_La\,(\beta C_s - C)$$

do/dt = oxygen uptake rate, mg/l/hr

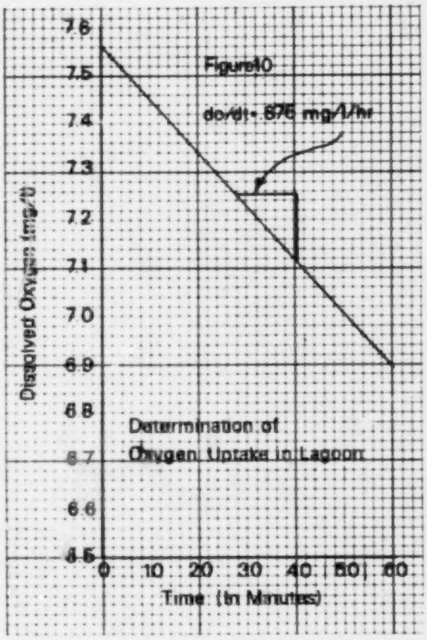
K_La = overall transfer coefficient, per hr.

C_s = Oxygen saturation in water

C = Oxygen concentration specified

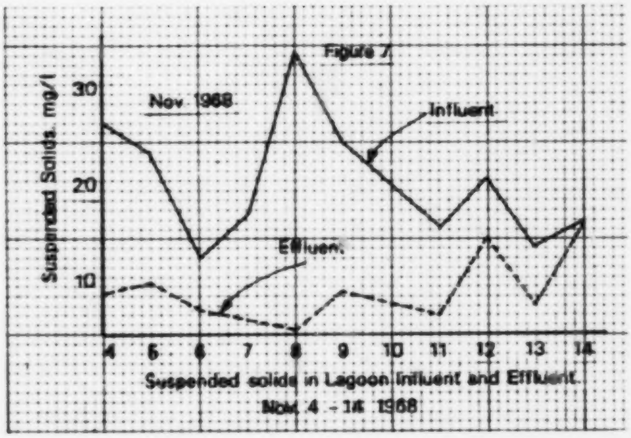
$$\beta = \frac{C_s \text{ of waste}}{C_s \text{ of water}}$$

Figure 10



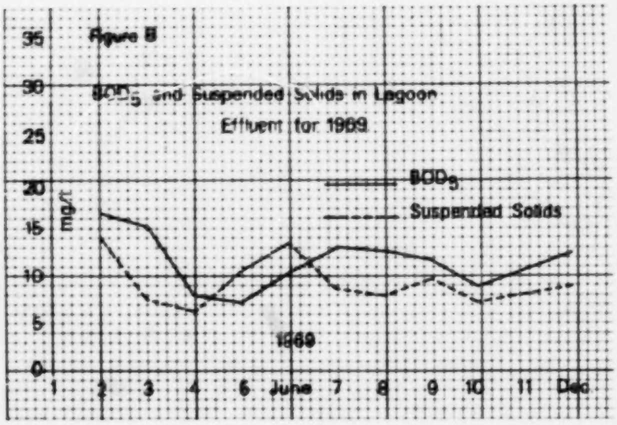
Determination of Oxygen Uptake in Lagoon

Figure 7



Suspended solids in Lagoon Influent and Effluent.
Nov. 4 - 14 1968

Figure 8



BOD$_5$ and Suspended Solids in Lagoon Effluent for 1969

Figure 11

Oxygen Transfer Equation

$$\frac{dc}{dt} = K_La\,(\beta Cs - C)(V)\,(8.34)(24)$$

dc/dt = oxygen transfer, lbs./day

c = 2.0 mg/l (specified)

v = volume of Lagoon (9.0 M.G.)

to the surface. Therefore it is recommended that steeper side slopes be used for lagoons of this type. Also, it is suggested that lagoons be deeper with smaller surface areas to minimize the opportunity for algal cell growth.

CONCLUSIONS

1. Oxidation ponds for the tertiary treatment of secondary plant effluent are not very effective in reducing BOD, suspended solids, and inorganic nutrients, unless a method of removing algae from the lagoon effluent is provided.
2. Aerated lagoons treating secondary treatment plant effluent can provide better than 60 per cent reduction of BOD and suspended solids. From data available no reduction in inorganic nutrients will occur through the aerated lagoon.
3. Side slopes for aerated lagoons should be steeper than 4:1, preferably 2:1, unless mixing on the slopes is provided to minimize the deposition of solids.

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A SIMPLIFIED KINETIC THEORY FOR AERATED LAGOONS

by
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and
M.J. Capri[3]

INTRODUCTION

Theories on the kinetic behaviour of aerated lagoons have been put forward by O'Connor & Eckenfelder in 1960(1) and McKinney in 1962(2). Marais & Shaw, in 1961(3), presented a theory on the kinetic behaviour of oxidation ponds which shows many points of similarity with the theories on aerated lagoons. The principal difference appears to be that whereas the aerated lagoon theories include the influence of the bacterial mass *per se*, in the oxidation pond theory it is incorporated indirectly by an all inclusive degradation constant. Such a simplification is of course only allowable if justified by experimental evidence.

Experimental data from O'Connor & Eckenfelder(1), McKinney *et al* (2)(9), which will be discussed later in the paper, appear to support the hypothesis that oxidation pond kinetics can be applied to aerated lagoons. Accepting this hypothesis, it is possible to extend the theory and investigate in a fairly simple manner the effects of series operation, recirculation, incremental feeding and equivalent pond systems, oxygen consumption, the effect of the sludge layer and the dynamic behaviour of the lagoon.

The basic precepts of the simple oxidation pond theory have been exhaustively discussed by Marais(4). Reference must be made to that paper for details, only the main conclusions will be presented here.

EQUILIBRIUM LAGOON BEHAVIOUR

Assume (i) complete mixing, and the mixing energy is sufficient to keep the settled solids in suspension; (ii) BOD reduction, (as measured for example by BOD5) takes place according to a first order reaction; (iii) the first order reaction constant is temperature-dependent according to the Streeter-Phelp approximation of the Van't Hoff-Arrhenius equation, i.e.

$$K_T = K_{T_0} \; \Theta^{(T - T_0)} \qquad \ldots\ldots \; (1)$$

where K_T, K_{tT_0} are the degradation constants in $\log_e$-day units at T and T_0°C respectively and 0 is a constant. It is assumed, further, that the value of K remains constant and equal for each lagoon in a series of lagoons.

Let P_i, P = influent and lagoon BOD5 in mg/l respectively
P_{ui}, P_u = influent and lagoon ultimate BOD (BODU) in mg/l respectively
V = volume of pond in gallons, cubic metres, etc.
Q = flow per day in same volume units as V
R = retention time in days = V/Q.

If there is no liquid loss, the differential equation of BOD change in a lagoon is given by:

$$dP/dt = P_i(Q/V) - KP - P(Q/V)$$
$$= P_i/R - (K + 1/R)P \qquad \ldots\ldots \; (2)$$

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The following solutions under equilibrium can be obtained:

Single pond

$$P = P_i/(KR + 1) \quad \ldots.(3)$$

which can be inverted to give a linear form:

$$P_i/P = KR + 1 \quad \ldots.(4)$$

Equation (3) was also derived by both O'Connor & Eckenfelder[(1)] and McKinney[(2)].

Series of ponds

Let R_1, R_2, ...R_n be the retention times in lagoons 1, 2, ... n in series. Then

$$P_n = P_i/[KR_1 + 1)(KR_2 + 1) \ldots.(KR_n + 1)] \quad \ldots\ldots(5)$$

If $\quad R_1 = R_2 = \ldots.R_n = R$

$$P_n = P_i/(KR + 1)^n \quad \ldots.(6)$$

Batch or plug flow conditions

In equation (5), let R.n = t = total retention time in a series of n lagoons with equal retention times, R. For a fixed value of t, as n tends to infinity, R tends to zero, and

$$P = P_i\ e^{-Kt} \quad \ldots.(7)$$

which represents the batch or plug flow process.

In equations (2) to (7) above, P_i and P can be replaced by P_{ui} and P_u, the ultimate influent and lagoon BOD's respectively. Strictly this interchange is only valid if the ratio of ultimate to 5-day BOD remains constant at every point in the processes. As this is unlikely, it generally introduces an error in the kinetic relationship.

Implications of equations (2) to (7) have been discussed by Marais[(4)] and may be summarized as follows: For the same total retention time, the completely mixed lagoon process is the least efficient and the batch process the most efficient; a series of lagoons, having the same total retention time as the batch and the single lagoon, has intermediate efficiency, the efficiency increasing as the number of ponds increases.

From the above, single lagoon and batch processes respectively represent the two extremes of pollution reduction that can be achieved. If it can now be shown that for a particular waste these two extreme processes [given by equations (3) and (7)] are approximately verified by experiment, then it can be reasonably inferred that intermediate processes exemplified by series of ponds [equation (5)] should also be approximately valid.

O'Connor & Eckenfelder performed experiments on batch and single lagoon processes using cannery and papermill wastes. In the lagoon experiments, particularly with the paper wastes, the temperatures differed from experiment to experiment. To use the experimental data effectively required that they be adjusted to a common temperature state (25°C) in accordance with the Streeter-Phelp's approximation, equation (1). After a number of trial attempts involving equations (3) and (1) simultaneously, adjustments using the constants in equation (8) gave the most consistent results:

$$K_T = 0.53\ (1.085)^{(T-25)} \quad \ldots.(8)$$

A plot of the adjusted experimental values (to the 25°C state) for the single lagoon, according to equation (4), is given in Fig. 1, which gives an experimental value of K_{25} approximately equal to 0.53. Plots of the adjusted values of both the single lagoon and batch processes, according to the method suggested by Marais & Shaw, are shown in Fig. 2. Also shown are the theoretical relationships [equations (3) and (7)] using constants at 25°C, [from equation (8)]. It is evident that there is a good correlation and, hence, one can deduce that it is very likely that the intermediate series lagoon process will similarly conform to the theoretical predictions.

Oxygen utilization

Oxidation utilization rate (OUR) of the lagoon contents may be derived as follows:

From the moment of sampling a lagoon the sample degradation will take place according to the batch condition and oxygen consumption, Y_t, is given by

$$Y_t = P_u\ (1 - e^{-Kt})\ \text{mg/l}$$

where t is the time measured from the moment of sampling and K is the lagoon process value.

The oxygen utilization rate is given by the rate of change of Y_t, i.e.

$$\text{OUR} = \frac{dY_t}{dt} = P_u K e^{-Kt}\ \text{mg/l} \quad \ldots\ldots(9)$$

at time, t = 0, OUR equals the rate of oxygen consumption in the lagoon:

$$\text{OUR}_{\text{lagoon}} = P_u K\ \text{mg/l} \quad \ldots\ldots.(10)$$

Total oxygen consumption for the pond volume, O_2, is given by

$$O_2 = P_u\ K\ V\ \Upsilon\ 10^{-6}\ \text{kg, lb/day} \quad \ldots\ldots(11)$$

where V = volume in m^3, gallons, ft^3

Υ = mass density of the volume unit in kg or lb.

An alternative expression for equation (11) can be derived as follows: From equation (3)

$$K = \left[(P_{ui} - P_u)/P_u\right]/R$$

Substituting in equation (11) and noting that Q = V/R,

$$O_2 = (P_{ui} - P_u)Q\Upsilon 10^{-6}\ \text{kg, lb/day} \quad \ldots.(12)$$

i.e.

$$O_2 = (\text{kg, lb of BOD removed/day})\ \text{kg, lb/day} \quad ..(13)$$

Normally the ultimate BOD is not known and equations (11), (12) and (13) are expressed in terms of BOD5. Assume $P_u = \propto P$, then

$$O_2 = \propto P\ K\ V\Upsilon\ 10^{-6}\ \text{kg, lb/day} \quad \ldots.(14)$$

$$O_2 = \propto (P_i - P)\ Q\ \Upsilon.\ 10^{-6}\ \text{kg, lb/day} \quad \ldots.(15)$$

$$O_2 = \propto\ (\text{kg, lb BOD5 removed/day})\ \text{kg, lb/day} \quad \ldots.(16)$$

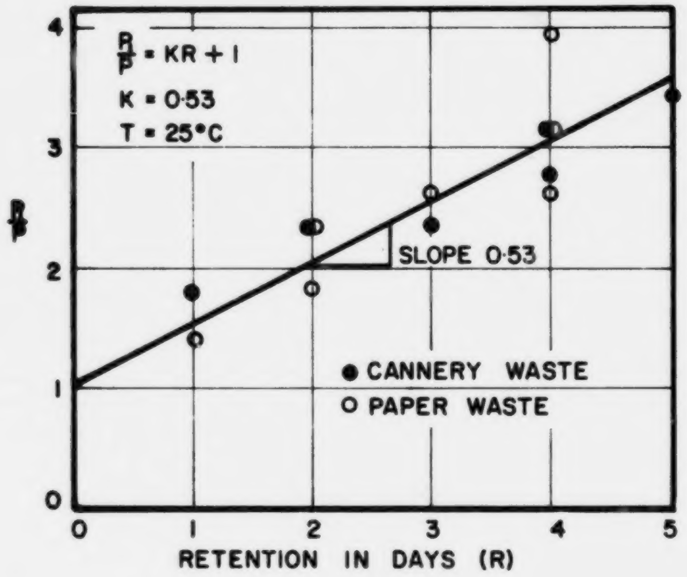


Fig. 1.

Determination of the degradation constant, K, in a single aerated lagoon. Data from O.Connor et al(1)

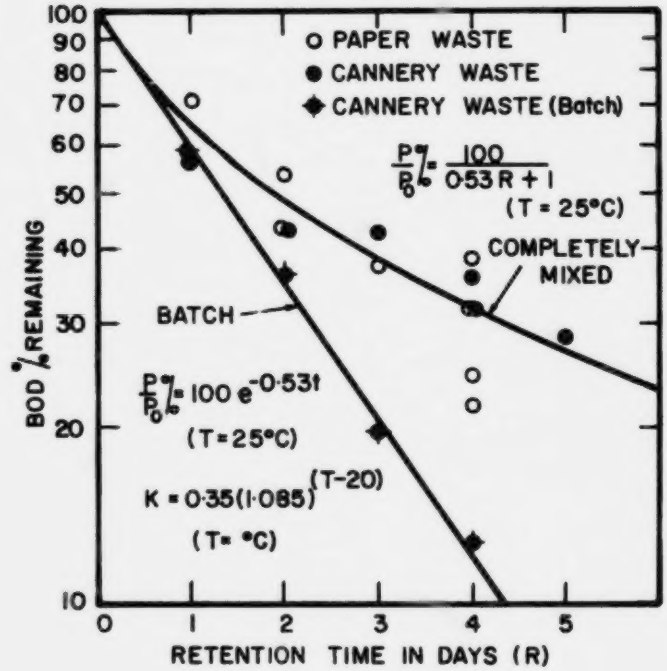


Fig. 2

Theoretical and experimental percentage BOD remaining for Paper and Cannery waste in a completely mixed single lagoon of varying(1) retention times and in a batch. Data from O'Connor *et al*

Equation (16) is the form used by O'Connor & Eckenfelder to determine the experimental value of α. Their data reflect a sufficient constancy in the value of α to be adequate for design purposes. The form expressed in equation (14) is useful when considering the dynamic behaviour of aerated lagoons.

Series operation

It was said earlier that for the same total pond volume (or total retention time) it is possible to achieve a greater reduction of BOD by subdividing the lagoon into two or more lagoons in series, or, expressed differently, for the same BOD reduction a smaller total series volume is necessary than for a single lagoon.

1. Optimum utilization of lagoon volume in series is achieved by making the ponds equal in volume: Let R_1, R_2 be the retention times of lagoons 1 and 2. Let $R_1 + R_2 = C$, hence $R_2 = C - R$.

From equation (5)

$$P = \frac{P_i}{(KR_1 + 1)(KR_2 + 1)} = \frac{P_i}{[K(C - R_1) + 1]\,(KR_1 + 1)}$$

Taking logs to base e and differentiating

$$\frac{1}{P}\left(\frac{dP}{dR_1}\right) = -\frac{K}{[K(C - R_1) + 1]} - \frac{K}{KR_1 + 1} = 0 \text{ for an extreme}$$

i.e. $2R_1 = C = R_1 + R_2$

i.e. $R_1 = R_2$

As the lagoons operate in series the volumes are also equal. The proof can be demonstrated for three or more lagoons in series.

2. For the same total BOD reduction, it is useful to determine the relationship between the total lagoon retention time in series and the retention time in a single lagoon

Let R_s= retention time in each lagoon in the series configuration (all the lagoons in the series assumed to have the same retention time)

R = retention time in the single lagoon configuration

Assume two lagoons in series; the effluent from the second pond is:

$$P_2 = P_i/(KR_s + 1)^2 \qquad \ldots\ldots(17)$$

$$= P_i/(KR + 1) \qquad \ldots\ldots(18)$$

From (17)

$$R_s = \left[(P_i/P_2)^{1/2} - 1\right]/K \qquad \ldots(19)$$

Substitute for P_i/P_2 from equation (18)

$$R_s = \left[(KR + 1)^{1/2} - 1\right]/K \qquad \ldots\ldots(20)$$

$$\text{Total series retention time} = 2\,R_s \qquad \ldots\ldots(21)$$

Equations (19) to (21) give the required relationships.

The effluent quality from each lagoon in the series is

$$P_1 = P_i/(KR_s + 1) = P_i/(Kr + 1)^{½}$$

$$P_2 = P_1/(KR_s + 1) = P_i/(KR + 1).$$

In general for a series of n lagoons, the relationships above become

$$R_s = \left[(KR + 1)^{1/n} - 1\right]/K \qquad \ldots\ldots\ldots(22)$$

or alternatively

$$R_s = \left[\left(\frac{P_i}{P_e}\right)^{1/n} - 1\right]/K \qquad \ldots(23)$$

where R = retention time in a single lagoon to give the same BOD reduction as in the series of n lagoons, each series lagoon having a retention time, R_s and P_e = effluent BOD from the series.

$$P_r = P_i/(KR + 1)^{r/n} \qquad \ldots(24)$$

where P_r = effluent BOD from the r^th^ lagoon, $r \leq n$

$$R_{Ts} = n\,R_s \qquad \ldots(25)$$

where R_{Ts} = total retention time in the series.

Example:

A cannery waste, BOD5 = 800 mg/l is to be treated in an aerated lagoon to give an effluent quality of 160 mg/l, i.e. 80% reduction. Lagoon temperature 20°C. Compare single lagoon with two lagoon series operation.

For cannery waste equation (8) gives $K_{20} = 0.35$. Single lagoon operation: From equation (18)

$$R = \left[(800/160) - 1\right]/0.35 = 11.4 \text{ days.}$$

Two lagoon series operation: From equation (23)

$$R_s = (800/160)^{½} - 1 \;\; /0.35 = 3.6 \text{ days.}$$

Total series retention time = 2 * R_s = 2 * 3.6 = 7.2 days.

Percentage volume reduction = 28%.

Reduction in retention time and volume requirements above hardly appear to justify the complication of two lagoons in series. However, where the reduction is in the region of 95% series operation appears more favourable. Assume in the example above 95% reduction in required and three lagoons in series are to be compared with single lagoon operation. Assume 500 000 American gallons/day flow and $\alpha = 1.1$(1). Single lagoon operation:-

$$R = [800/40 - 1]/0.35 = 54 \text{ days.}$$

Three lagoon series operation

$$R_s = \left[(800/40)^{1/3} - 1\right]/0.35 = 54 \text{ days}$$

Total series retention time = 3 * 4.9 = 14.7 days

Percentage volume reduction = 70%

Other parameters of interest are:-

Parameter	Single lagoon	Series 1	lagoons 2	3
Effluent BOD5 mg/l	40	295	108	40
O_2/day in lb	3500	2360	840	300
Area in acres	8.8	0.78	0.78	0.78

In the single pond facility, because of the massive volume, the mixing energy imput to keep the organic solids in suspension, will probably exceed the energy imput to supply the oxygen requirements. If only the oxygen energy requirements are provided, settlement will occur of the organic solids and bacterial floc. Dostal(5) found that if the mixing energy was insufficient, floc settlement reduced the BOD degradation rate and foam problems increased. With series operation, in the first lagoon mixing requirements will probably be sufficient when aeration requirements are met; additional mixing energy need only be supplied in the second and third lagoons and here the volumes to be mixed are considerably less than in the equivalent single lagoon.

In practical application the relationships derived above will probably be only approximately applicable. The implication is however clear that series operation should lead to improved utilization of lagoon volume.

Incremental feeding

A system of parallel lagoons, each lagoon receiving the same unit volumetric waste loading, delivers the same quality effluent as the identical lagoons in series in which the loading is applied incrementally, as shown in Figures 3(a) and 3(b). For simplicity, the proof will be given for a set of n identical lagoons each receiving the same volumetric load.

Referring to Figure 3(a) the effluent from the set of parallel lagoons is given by

$$P_1 = P_2 = P_3 = P_i/(KR + 1)$$

P_1, P_2 .. P_n = effluent BOD from lagoons 1, 2, .. n respectively.

V = volume of each lagoon in 3, gallons, etc.

Q = raw sewage BOD flow to each lagoon in same units per day as V

R = retention time in days = V/Q

K = first order degradation rate in $\log_e$-day units.

Referring to Figure 3(b), mean BOD, P_{av}, in the effluent of the n^{th} lagoon is:

$$P_{av} = \left[P_i Q + (n-1) P_{(n-1)} Q\right]/(nQ)$$

If R = V/Q, the effluent from the n^{th} lagoon is given by:

$$P_n = P_{av}/\left[KV/(Qn) + 1\right]$$

$$= \frac{P_i + (n-1) P_{(n-1)}}{KR + n} \qquad \ldots\ldots (26)$$

It will now be proved that:

$$P_n = P_i/(KR + 1) \qquad \ldots\ldots\ldots (27)$$

Statement 1 Equation (27) is true for n = 1:

Substituting n = 1 in equation (26) and reducing

$$P_1 = P_i/(KR + 1) \qquad \ldots\ldots (28)$$

Statement 2 If equation (27) is true for lagoon (n - 1) then it is true for lagoon n: As it is assumed that statement 2 is true for lagoon (n - 1)

$$P_{(n-1)} = P_i/(KR + 1) \qquad \ldots\ldots (29)$$

Substitute equation (29) in equation (26) and reducing

$$P_n = \frac{P_i(KR + 1) + (n - 1) P_i}{(KR + 1)(KR + n)}$$

$$= P_i/(KR + 1)$$

Hence true for lagoon, n, but it is true for n = 1 [equation (28)]. From statement 2 it is then true for n = 2, hence true for n = 3 . . . ., hence true for all n.

It can be demonstrated that the theorem is also true if the lagoons have different volumes provided their nominal retention times, i.e. $V/Q_{raw\ sewage}$ values are the same. Thus, instead of constructing a parallel lagoon, by incremental feeding to the first and second ponds in a series the same effect as an additional parallel primary pond can be achieved.

Inter-lagoon recirculation

Liquid is cycled from one of the subsequent lagoons in the series to the primary lagoon.

Based on the simple kinetic model discussed previously, Marais & Shaw(3) deduced equations which give guide lines on the behaviour of an inter-lagoon recirculation system.

Referring to Figure 4, consider a series of lagoons, each lagoon in the series having a different retention time. Effluent is circulated from the n^{th} lagoon, not necessarily the last in the series, to the primary lagoon.

Let Q_1 = influent flow of raw sewage to the primary lagoon

P_i = BOD 5 in influent to series

P_n = BOD5 in n^{th} lagoon

a = recirculation ratio defined by $Q_{recirc.} = aQ_1$

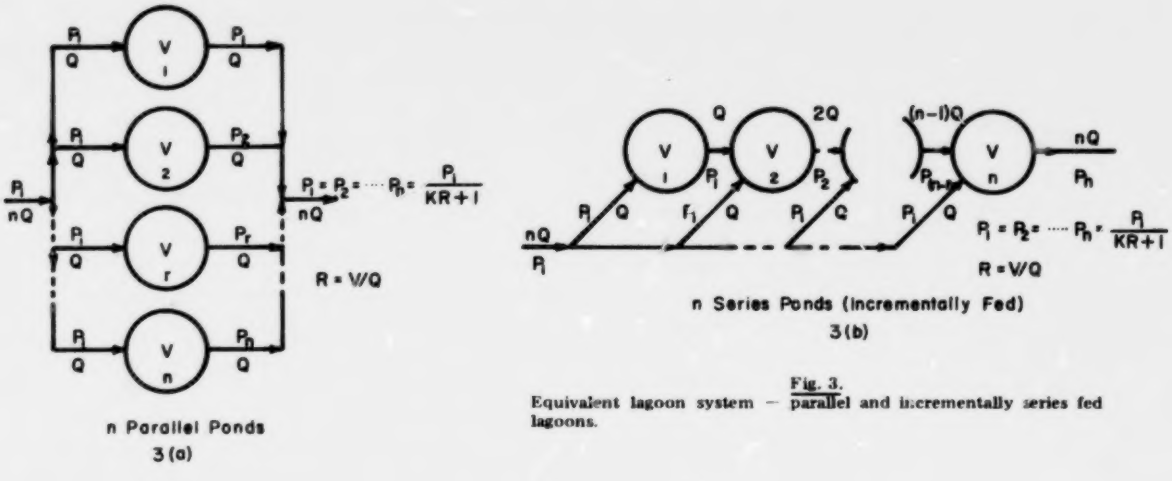
V_n = volume of the n^{th} lagoon measured in the same units as the flow

r = lagoon number between 1 and n

and R_n = inflow retention time in days in the n^{th} lagoon defined by V_n/Q_1.

Assume no evaporation or seepage losses. Recycling is from the n^{th} to the primary lagoon. The BOD in the n^{th} lagoon is determined by:

$$P_n = P_i/\left[\frac{\prod_{1}^{n} (KR_n + a + 1)}{(1 + a)^{n-1}} - a\right] \qquad \ldots\ldots (30)$$



n Parallel Ponds
3(a)

n Series Ponds (Incrementally Fed)
3(b)

Fig. 3.
Equivalent lagoon system — parallel and incrementally series fed lagoons.

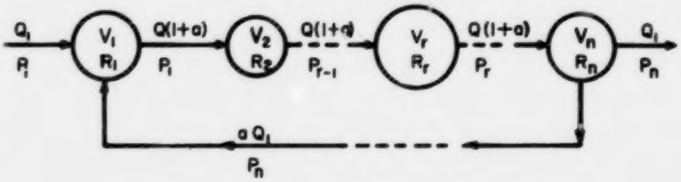


Fig. 4.
Series lagoon system with interpond recirculation.

BOD in lagoons 1, 2, ... r for r smaller than n is then found to be:

$$P_r = \frac{(P_i + aP_n)(1 - a)^{r-1}}{\prod_{1}^{r} (KR_r + a + 1)} \quad \text{.. (31)}$$

In Figure 5 the effect of recycling at different rates, a, in a series of three equal lagoons (recycling from the 4th to the first pond) is illustrated. The value of K is taken equal to 1.0 for illustrative purposes. The values are plotted against the nominal total retention times, i.e. R is based on the raw influent flow. The diagram shows that the pollution in the primary lagoon is reduced but increased in the final lagoon.

The sludge layer

When the mixing energy is insufficient to keep the settleable solids in suspension, a sludge layer builds up in the bottom of the lagoon. This has been observed by Eckenfelder(1) and McKinney(2). The latter found a 6 inch deposit of organic material after 6 months operation in a pond treating domestic wastes.

Anaerobic fermentation develops in the layer, which releases nutrients to the supernatant, gas to the atmosphere and heat energy. The gas, a mixture principally of CO_2 and CH_3, still contains bound chemical energy and its loss, therefore, relieves the supernatant of this fraction of the influent load energy. The sludge effect can be incorporated into the degradation rate equations if the following assumptions are made:-

(1) Sludge fermentation is a first order reaction, the rate K_s being temperature dependent as follows:

$$K_s = K_{sTo}\, \theta_s^{(T - T_o)} \quad \text{....(32)}$$

(2) All BOD's are ultimate first stage.

(3) Complete and instantaneous mixing in the lagoon liquid and no liquid loss.

(4) A fraction, i_p of influent BOD, P_{ui}, is dispersed within the body of the lagoon, the remaining fraction $(1 - i_p)$ settles as sludge.

(5) A fraction, s_p, of BOD lost in the sludge fermentation enters into the lagoon liquid, the remaining fraction leaves the system as heat energy and fermentation gas.

Let S_t = total sludge mass in terms of BODU in kg or lb. Rate of change of sludge mass:

$$\frac{dS_t}{dt} = (1 - i_p)\, P_{ui} Q\, \Upsilon\, .10^{-6} - K_s S_t \quad \text{.... (33)}$$

Rate of change of BODU in lagoon:

$$\frac{dP_u}{dt} = i_p\, P_{ui}/R - (K + 1/R)P_u + s_p\, K_s\, S_t/(V\Upsilon).\ 10^6 \quad \text{..(34)}$$

If K and K_s remain constant, then at equilibrium

$$S_t = (1 - i_p)\, P_{ui}\, Q\Upsilon\ 10^{-6}/K_s \text{ kg, lb} \quad \text{....(35)}$$

$$P_u = P_{ui}\,(i_p + (1 - i_p))/(KR + 1) \text{ mg/l} \quad \text{...(36)}$$

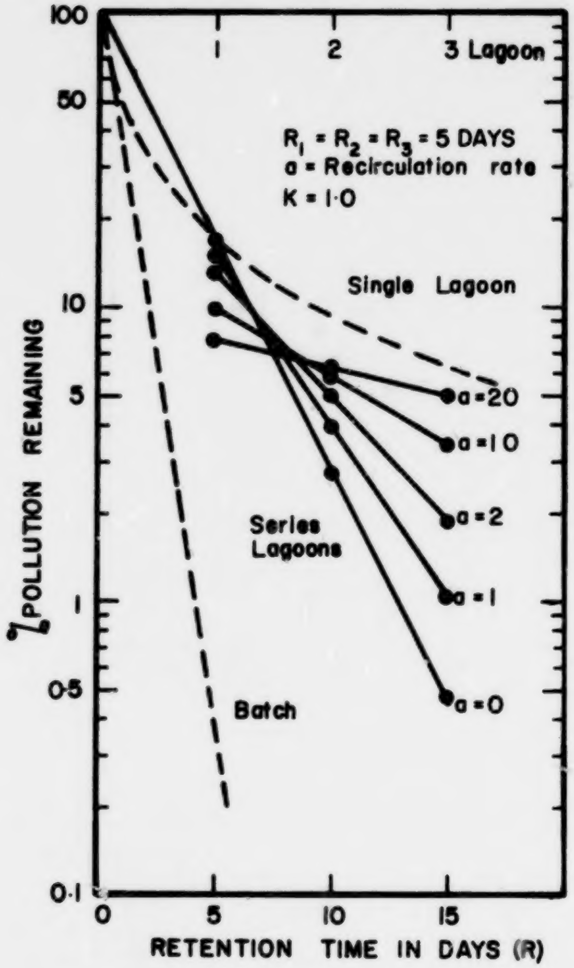


Fig. 5.

Influence of recirculation rate on the concentration of BOD in series of 3 equal ponds. Recirculation from lagoons 3 to 1.

Although i_p and s_p are assumed constant the relative effects of the fermentation products and nonsettleable influent BOD on the lagoon BOD change significantly over the yearly temperature cycle. The reasons for this are that equilibrium is never established under cyclic changing temperature conditions and K_s and K are not temperature dependent to the same degree. The fraction i_p is also dependent on the mixing energy imput to the pond, equal to one at high and probably near 0.4 at no energy imput. Mainly because of the temperature effect on K_s and K during cyclic temperature changes, there is little merit in considering equilibrium conditions any further.

Non-uniform degradation

In the discussion above the value of K was considered to be only temperature dependent. In a waste having fractions differing in ease of degradation, the process may reflect this by having a decreasing value of K with retention time. Consequently, in a series of lagoons the reduction in the primary lagoon may indicate a larger value of K than in the secondary lagoons. However there is one point should always be remembered. If the data from ponds are used to determine K from, for example equations (3) and (5), it may in fact lead to the wrong deduction that K decreases with retention time because the equations do not include the effect of the sludge layer. If the energy imput is sufficient to prevent settlement and a decreasing K is still evident, an analysis incorporating a decreasing K value can be made according to the method set out by Vincent, Algie & Marais(6). Briefly, the differential equation for the lagoon becomes

$$\frac{dP}{dt} = P_i/R - P/R - K(P/P_i)^n P$$

where n is Fair's(7) coefficient of non-uniformity.

Equilibrium solutions are as follows:-

(i) Single lagoon

$$P = P_i/\left[K\,(P/P_i)^n R + 1\right]$$

(ii) Series of lagoons

$$P_m = P_i/\left[\prod_1^m (K(P_m/P_i)^n R_m + 1\right].$$

where the subscript, m refers to the m^{th} lagoon.

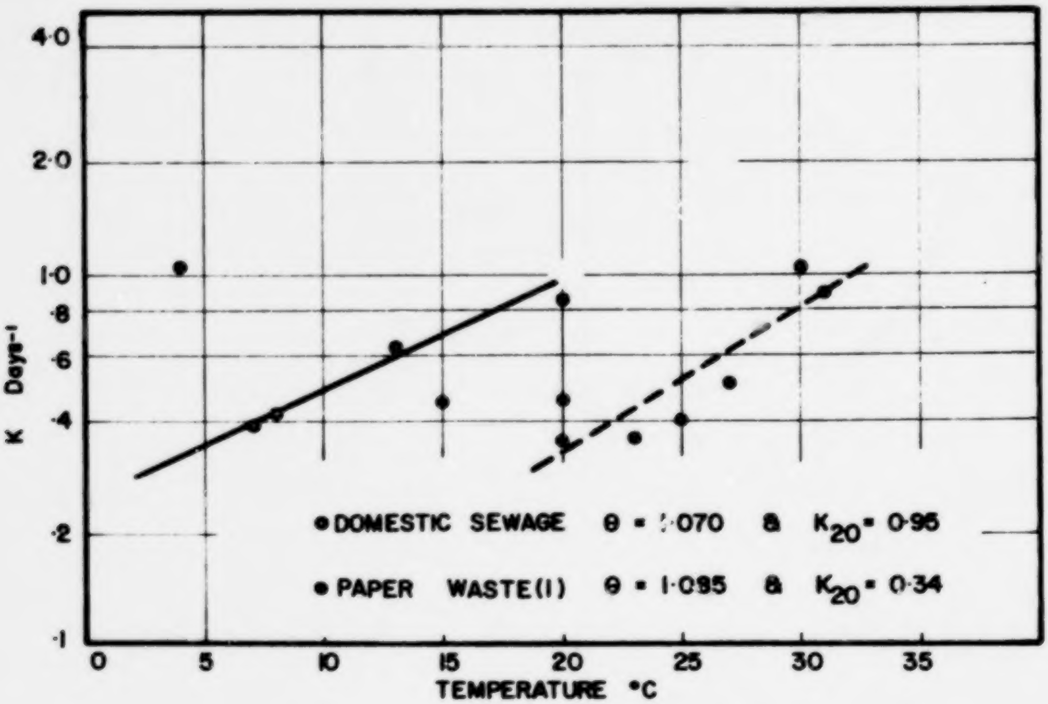


Fig. 6.
Effect of temperature on the degradation rate, K, in single lagoons for domestic and papermill wastes. Data from Eckenfelder(1) (11)

(iii) Plug flow

$$P = P_i (1 + nKt)^{-1/n}$$

When $n = 0$ the equations reduce to the corresponding equations for a fixed value of K.

The first two equations are particularly difficult to solve as P, P_m occur implicity in the solutions. However a simple graphical method to solve the equations and to determine n from experimental data is described by Vincent *et al*(6).

DYNAMIC BEHAVIOUR OF AERATED LAGOONS

If i_p, s_p, θ, θ_s, K and K_s are known the behaviour of a lagoon over the yearly temperature cycle can be simulated by solving equations (33) and (34). Information on these parameters is meagre. Estimates can be obtained by reference to data by Marais(4)(8) on oxidation pond sludge behaviour, O'Connor & Eckenfelder(1) on K values for lagoon liquids at different temperatures, and McKinney & Benjes(9) on BOD's and oxygen consumption over yearly cycles of temperature in lagoons.

Marais(4) obtained rate constants for the sludge layer and a value of s_p in oxidation ponds.

$$K_s = 0.002\,(1.35)^{(T-20)} \qquad \ldots\ldots(37)$$

$$s_p = 0.4 \qquad \ldots\ldots(38)$$

Eckenfelder(11) listed some data obtained from domestic wastes treated in aerated lagoons at different temperatures. A Plot of the data is shown in Figure 6, from which the value of θ and K are estimated:

$$K = 0.35\,(1.07)^{(T-20)} \qquad \ldots(39)$$

The value of K will of course change with the waste being treated. In the case of industrial wastes it is therefore necessary to run laboratory-scale experiments to determine the kinetic rate and its temperature dependence.

McKinney & Benjes(9) reported BOD, flow, temperatures and oxygen consumption data on two aerated lagoons treating domestic wastes. Their data are taken as a basis for testing the theoretical model for aerated lagoons described above. Preliminary estimates of K and θ, K_s and θ_s are taken from equations (39) and (37), Ultimate BOD's are assumed 1.47 times the 5-day values. The ponds are simulated by step-by-step solutions of equations (33) and (34) using the imput values of temperature, flow and BOD reported by McKinney. The effluent BOD and oxygen concentrations are calculated from the lagoon and sludge equations.

The method of numerical integration is described by Marais(4) and reference must be made to that paper for details. The experimental determinations reported are at unequal time intervals and Marais's program must be modified to incorporate this. The discrete forms of the differential equations are given in Appendix A. The forms are of second order accuracy and developed by Marais & Shaw based on the work of Kopal(10).

It is necessary to use a form giving high accuracy as the time intervals are often large - up to 4 weeks.

A number of simulations were carried out varying the constants until the best concurrence between experimental and simulated data for effluent BOD5 values and oxygen consumption rates were obtained.

In Figure 7 the experimental and simulated data are compared for Red Bridge Lagoon No.2. This lagoon was operated continuously over a year but readings were

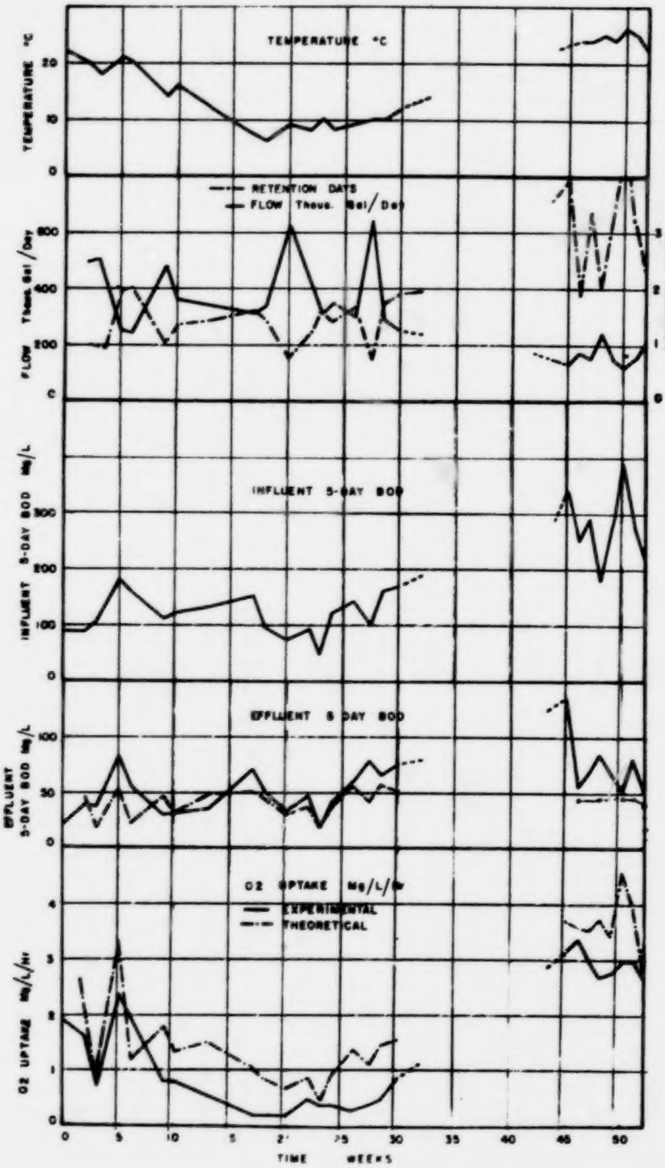


Fig. 7.

Simulated and experimental BOD effluent quality and oxygen consumption rate in Red Bridge No. 2 aerated lagoon. Data from McKinney & Benjes(9)

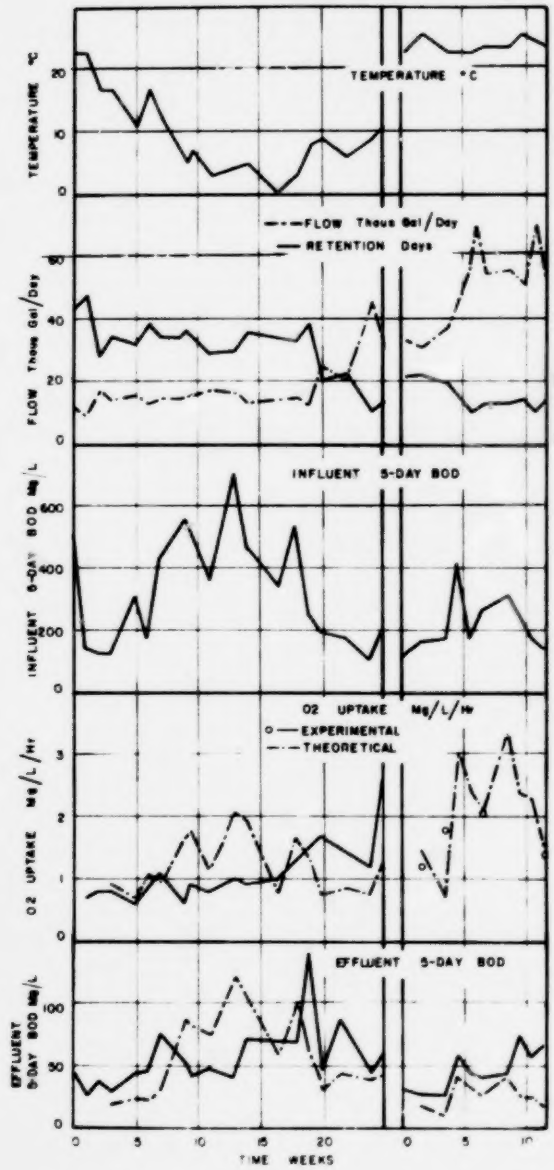


Fig. 8.
Simulated and experimental BOD effluent quality and oxygen consumption rate in Red Bridge No. 1 aerated lagoon. Data from McKinney & Benjes(9)

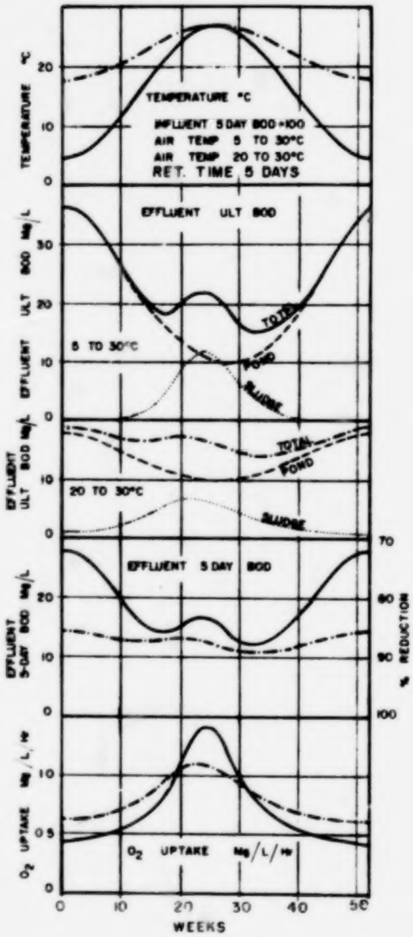


Fig. 9.
Simulated behaviour of two lagoons with the same retention time under different yearly cyclic temperature changes.

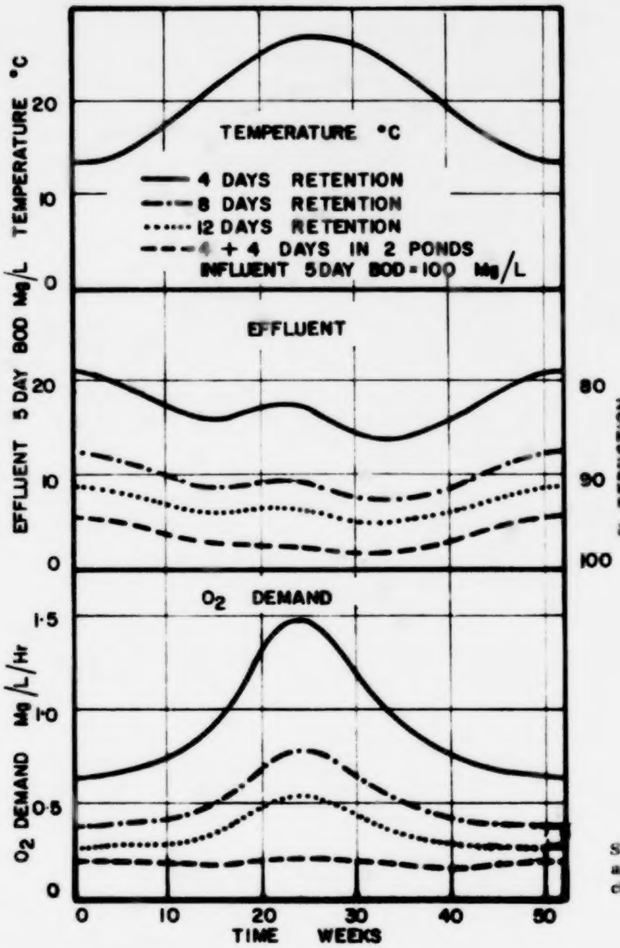


Fig. 10.
Simulated behaviour of single lagoons with different retention times and lagoons in series under the same yearly cyclic temperature change.

discontinued between April and July. During this period values for imput data were estimated conforming to the trends preceding and following the period. Effluent data, during the second summer period, are unreliable as the aerator was not capable of supplying sufficient oxygen to keep the lagoon aerobic.

Figure 8 compares experimental and simulated data for Red Bridge Lagoon No.1. In this lagoon readings were taken over the first six months of operation after which the lagoon was emptied. The lagoon was then enlarged and it is assumed that the sludge which had accumulated was removed. Experimental readings were taken for the first three months after the lagoon was put back into operation.

From the two simulations, using various combinations of constants, the best overall fit for both lagoons was obtained by using the following constants:

Lagoon liquid
$K_{20} = 0.95$, $\theta = 1.080$, $i_p = 0.6$
Sludge layer
$K_{20} = 0.002$, $\theta = 1.35$, $s_p = 0.4$

The values for the sludge layer are the same as those used by Marais[(4)] in his oxidation pond simulation. The longest period of continuous operation after the lagoons were put into operation was one year, so that the sludge layer fermentation played a relatively minor part during the period of high fermentation in the summer. It is not possible therefore to obtain improved values - this would require experimental data over a number of years. The kinetic constants for the lagoon liquid appear to be within the ranges reported by Eckenfelder[(11)], i.e. $\theta = 1.085$ to 1.1, $K_{20} = 0.5$ to 1.0. The value for i_p, the fraction of influent BOD to the pond liquid, although higher than for an oxidation pond ($i_p = 0.4$), still indicates inadequate mixing energy imput to ensure suspension of the organic solids.

In the experimental lagoons above the influent BOD and flow, and temperature exhibit wide fluctuations and seasonal trends of the lagoon behaviour are not specific. To obtain a clearer picture of seasonal temperature effects, a number of simulations were performed on lagoons with different retention times, in which influent BOD and flow were kept constant and the temperature varied sinusodially over the year. The final constants listed above were used and the simulation continued for a period of 5 years, the fifth year being reported here. (Figure 9). The effect of the sludge layer is now clearly evident. During the cold season little degradation takes place in the sludge and the sludge is simply stored in the pond. During the hot season the products from the increased fermentation of the sludge accumulated during the cold season are offloaded to the supernatant lagoon.

The degradation rate in the lagoon increases during summer and a low equilibrium lagoon BOD is established in the pond. However the low value is raised by the products of fermentation diffusing into the liquid above. The effect of sludge fermentation is therefore to tend to equalize the summer and winter lagoon BOD values.

The rate of oxygen demand [given by equation (10)] is significantly increased during summer. This is due to the increase in the lagoon degradation constant K even though the lagoon BOD is at its lowest value. Rate of oxygen requirements may increase up to 5 times from winter to summer.

To investigate the effect of different retention times the same cyclic variation of temperature was assumed and lagoons with 4, 8, 12 days retention and a series of two lagoons with 4 + 4 days retention simulated. (See Figure 10). The second lagoon in the 4 + 4 day series was assumed not to have any sludge settlement. The increase in BOD reduction in the 4 + 4 day lagoon series over the 8-day lagoon is clear. The absolute variation of BOD in the second lagoon of the series is small and the oxygen demand is virtually constant.

CONCLUSION

From their inception, because of their long retention times, aerated lagoons have been of the completely mixed single reactor basin type. In contrast the earlier designs of activated sludge plants were essentially of the plug flow type; the advantages of completely mixed single reactor chamber operation has become evident relatively recently. Plug flow and completely mixed basin types are two extreme configurations, and it seems that the time has come to give serious consideration to the intermediate configurations represented by series operation of reactors.

In this paper an attempt has been made to point out the theoretical advantages in operating aerated lagoons in the series configuration. The basis of the investigation is that the process degradation action is approximately first order. The derived relationships should form a framework for further research investigation, design and operation procedures.

The various lagoon configurations, feed and operating procedures,i.e. series and parallel configuration, series incremental feeding, and interpond recirculation lend flexibility to lagoon loading capacity and operation. For example, to keep the BOD in the effluent at a relatively even value between winter and summer, series operation during winter is indicated because K is low; parallel and incremental feeding during summer because K is high, and/or when the load is high. To accommodate temporary overloading, interpond recirculation will assist in reducing the load on the first lagoon. The situations that can arise are numerous and each one requires specific consideration for an optimum solution.

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ANAEROBIC LAGOONS - THEORETICAL CONSIDERATIONS

by
John T. Pfeffer

INTRODUCTION

The anerobic lagoon has been used in many applications, both in the treatment of municipal and industrial wastewaters. The simplicity of operation is a decided advantage over conventional treatment processes. The land requirements are substantially less than those of the traditional oxidation pond. The microbiological processes employed in these systems are essentially the same as those employed in the anaerobic digestion and the anaerobic contact process. The major difference is the lack of control of the environmental parameters associated with the anaerobic lagoon.

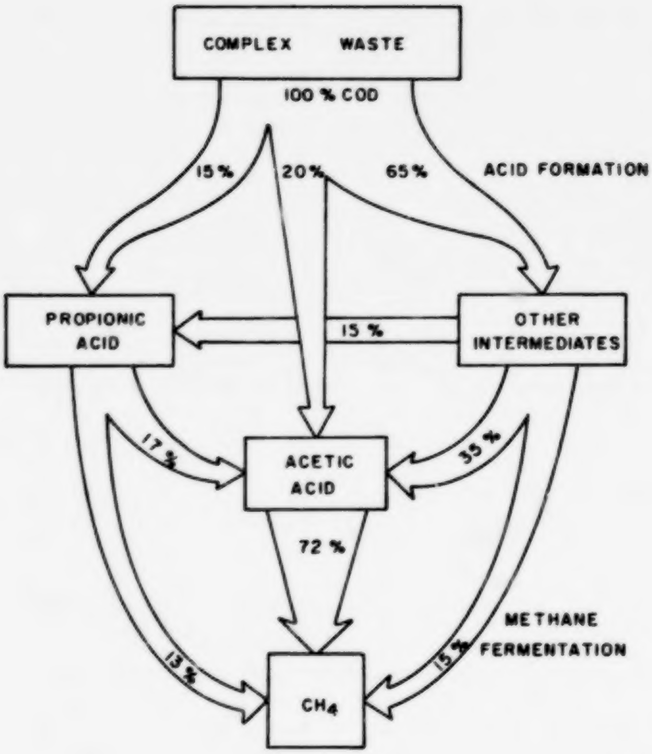

Figure 1. Methane fermentation of a complex organic waste, after McCarty (1).

The more sophisticated anaerobic processes are contained in enclosed units. This allows for positive control of the environment. The systems have external heating for temperature control. The enclosed tanks assure exclusion of oxygen as well as containment of odoriferous gasses. Also positive mixing devices are employed to prevent stratification of the contents with the associated problems. These controls are not applied to the anaerobic lagoon.

An understanding of the basic mechanisms involved in the anaerobic process will markedly assist in design of lagoons that will function as intended. These systems have definite applications and when properly employed can provide an effective wastewater treatment system at low costs.

BIOCHEMICAL TRANSFORMATIONS

The stabilization of organic material by anaerobic digestion is basically a two-stage mechanism. This is best shown by Figure (1). While this figure is oversimplified, it presents the basic relationship that must exist in anaerobic digestion. The incoming complex organics, consisting of proteins, carbohydrates, and fats undergo acid fermentation. This converts about 35 percent of the organic material into shortchain organic acids, i.e., propionic and acetic. The other 65 percent of the waste is converted to alcohols, aldehydes, and long-chain fatty acids. These percentages depend upon the waste composition and will vary markedly for wastes containing different organic compounds. Regardless of the percentage relationship, this transformation must be made before methane fermentation can proceed.

In this initial acid-fermentation stage, there is very little stabilization of organic material. This is often called the constant-BOD stage, because all that occurs is the chemical rearrangement of the organic molecules. The acceptors of the hydrogen from the oxidized compounds are organic compounds that have higher oxidation states. These hydrogen acceptors are reduced as a result. In most cases, the energy yield from this transformation is very low and consequently the microorganism growth is low. The energy loss from the system is also low because most of the energy yielded by the bacterial metabolism is used for synthesis of organic material in the form of bacterial protoplasm.

In the second stage shown in Figure 1, the end products of the first stage undergo methane fermentation. In this stage, carbon dioxide acts as the hydrogen acceptor and is reduced to methane gas. It is the production and release of methane gas that results in stabilization of the organic material. Methane is insoluble and consequently is lost from the system. The oxygen equivalent of the methane is given by the following equation:

$$CH_4 + 2\,O_2 \rightarrow CO_2 + 2H_2O \qquad \ldots\ldots\ldots (1)$$

One mole of methane requires two moles of oxygen to oxidize it to carbon dioxide and water. Each 1,000 cu ft of methane produced at STP will remove 178 lb of COD or ultimate BOD. This is the means for achieving stabilization of the organic material.

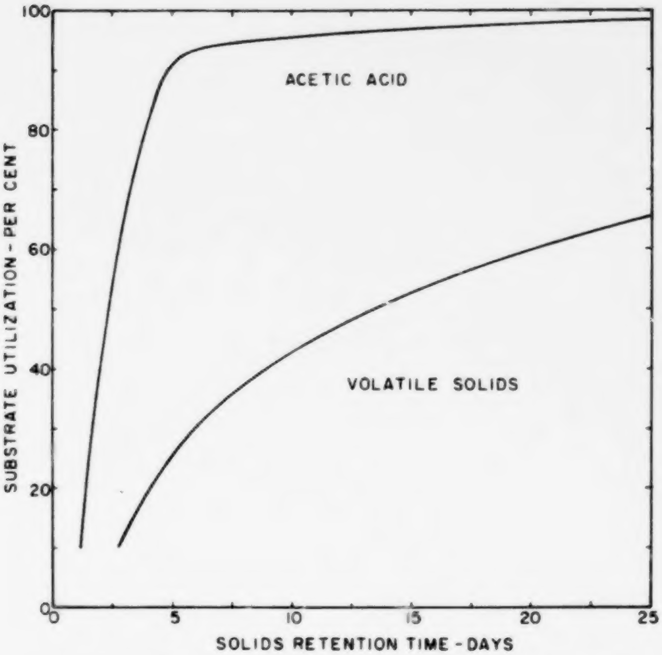

Figure 2. The relationship between solids retention time and substrate utilization.

The anaerobic stabilization system is one of the most complicated in terms of the different microorganisms involved in converting the complex organics to methane. In order to understand the kinetic relationships in anaerobic digestion, the rate-limiting step must be ascertained. McCarty (2) investigated this point and showed that the rate-limiting step appeared to be the metabolism of lipids and short-chain fatty acids. Heukelekian (3) has shown that the lipids in municipal sludge consist mainly of triglycerides of fatty acids. McCarty's investigations showed that the hydrolysis of the lipids to long-chain fatty acids was not rate limiting, but the acid metabolism was. Since both long-chain and short-chain fatty acids are degraded by methane fermentation, this is the rate limiting step in most digestion processes.

Generally when an anaerobic process is loaded at a high rate, volatile organic acids increase in the system. The design of the process requires that this increase in organic acids does not occur. As long as the system can be operated such that these acids are fermented as soon as they are produced, the process will function effectively without any operational problems. Therefore, retention times are generally srate limiting.

A typical relationship between the utilization of acetic acid and retention time has been taken from Lawrence and McCarty's work (4). This is shown in Figure 2. At a 10-day solids retention time and 35°C, 95 percent of the acetic acid has been utilized; at 5-days, 91 percent of the acid was utilized. Therefore, a 10-day retention time in anaerobic digestion would be satisfactory. However, at this retention time, volatile solids destruction is only about 43 percent complete. These data were collected at 35°C for a combination of primary and waste activated sludge (5,6). Increasing the solids retention time to 25 days increases the volatile solids destruction to 66 percent. Therefore, the rate limiting step in anaerobic digestion has been changed. In a system operating with a solids retention time of approximately 10 days at 35°C, the rate limiting step is methane fermentation. When the solids retention time is much greater than about 10 days, the rate limiting step then becomes hydrolysis of the organic solids, i.e., acid fermentation.

The significance of the two rate-limiting conditions is dependent upon the characteristics of the wastewater. If the organic material is readily biodegradable, then the methane fermentation will be rate limiting. The organic material is quickly hydrolysed to organic acids. However, if the wastewater contains significant quantities of more resistant material, the hydrolysis of this material will be rate limiting. Substantially longer times will be required to obtain the same degree of waste stabilization.

The lack of mixing in the anaerobic lagoon is advantageous in that the solids that may be more biologically resistant can settle to the bottom. The time available for decomposition of these materials is substantially greater than the flow-through time. The readily degraded material can be quickly stabilized via methane fermentation. The solids deposited on the bottom can be slowly hydrolysed and then fermented to methane.

MICROORGANISMS

Limited information is available concerning the predominate organisms associated with the acid formation steps in the anaerobic process. They undoubtedly range from facultative organisms that can anaerobically ferment simple carbohydrates, to strict anaerobes capable of converting complex proteins and carbohydrates to organic acids. The reactions of many of these organisms are known, particularly as related to the fermentation industries. It is known that the end products of carbohydrate and protein fermentation will depend upon the species of

TABLE 1
Classification of Methane Bacteria*

- A. Rod-shaped cells
  - I. Non-sporulating: *Methanobacterium*
    - 1. *Mbact. formicicum*: formate, carbon monoxide, hydrogen
    - 2. *Mbact. propionicum*: proprionate
    - 3. *Mbact. söhngenii*: acetate, butyrate
  - II. Sporulating: *Methanobacillus*
    - 1. *Mbac. omelianskii*: primary and secondary alcohols, hydrogen
- B. Spherical Cells
  - I. Cells not in sarcina arrangement: *Methanococcus*
    - 1. *Mc. mazei*: acetate, butyrate
    - 2. *Mc. vannielii*: formate, hydrogen
  - II. Cells in sarcina arrangement: *Methanosarcina*
    - 1. *Ms. barkerii*: methanol, acetate, carbon monoxide, hydrogen
    - 2. *Ms. methanica*: acetate, butyrate

* After Barker (10)

microorganisms involved. For example, with glucose fermentation, the end product may be ethyl alcohol, lactic acid or acetic acid, depending upon the organisms utilized. Since no control is exercised over the predomination of the organisms in anaerobic digestion, it could be expected that shifting populations of acid-forming microorganisms can result in the production of different organic acids at different times

There has been a considerable amount of work done on the rumen organisms. Since there is a similarity between the rumen and the anaerobic process, it could be expected that many of the same organisms could be found in both systems. The major difference may be in the substrate composition. Bryant (7) summarized the significant aspects of rumen bacteria. The end products of fermentation of the various foods found in the animal's diet are volatile fatty acids, carbon dioxide and some methane.

Another important group of microorganisms found in the rumen, but seldom reported in the anaerobic process, are protozoa. Gutierrez and Davis (8) summarized the activities of these organisms. The major function of protozoa is the digestion of substrates such as starch and cellulose as well as the bacteria. They also are capable of breaking down proteinaceous materials in the substrate as well as metabolizing long chain fatty acids such as oleic and lauric. These rumen protozoa are responsible for the production of about 20 percent of the total volatile acids available to the host. These organisms could be found in certain anaerobic processes receiving wastewater high in complex carbohydrates and proteins. Hungate (9) has recently published a text on rumen microbiology that is very useful in studying the microbial activity in the rumen and the relationship to the anaerobic process.

The methane producing organisms found in the anaerobic process have been more thoroughly studied. There are several different species of these organisms, but all are strict anaerobes. The common characteristic is their ability to produce methane from simple organic compounds under anaerobic conditions. These organisms are specific in their substrate requirements and can ferment only a relatively restricted group of simple organic compounds.

Most of the studies of the properties of these organisms have been conducted on enriched cultures rather than pure culture conditions. Table 1 shows the classification of methane bacteria and their substrate requirements, as indicated by Barker (10). The restricted number of substrates each organism is capable of utilizing suggests that several species would be present in any digestion process treating a complex waste. Studies (11, 12) on the microorganisms present during anaerobic metabolism of simple, as well as complex substrates, have indicated organisms with characteristics similar to those reported by Barker (10).

HYDROGEN ACCEPTORS

The hydrogen acceptor generally associated with methane fermentation is carbon dioxide. In the process of metabolism of the organic acid, hydrogen is transferred to the carbon dioxide producing methane and water. The methane; being insoluble is released from the liquid resulting in a reduction of the oxygen demanding potential of waste. Each pound of methane released results in a reduction of the oxygen demanding potential of the system by 4 pounds.

There are other hydrogen acceptors that can play an important role in anaerobic processes. The presence of oxygen is detrimental to the process, not because of the role of a hydrogen acceptor, but because of its toxicity. The presence of any oxygen in the digestion liquid will drastically inhibit the methane fermentation. Oxidized forms of nitrogen have similar effects. This toxicity is probably related to the oxidation-reduction potential of the system. Methane fermentation has been reported to be more active at E_c values of -520 to -530 mv. (13). These ORP levels are not commensurate with free oxygen or oxidized forms of nitrogen.

The role of sulfate as a hydrogen acceptor is more complex. The sulfate present in the carriage water or added to the waste stream will serve as a hydrogen acceptor for microorganisms such as *Desulfovibrio*. The end product of this reaction is hydrogen sulfide. Not only does the use of this hydrogen acceptor reduce the methane generation, but it also produces an odoriferous gas. This sulfide can cause a toxic condition to develop in the digestion process. This will be discussed in more detail.

The energy obtained from the reduction of the different hydrogen acceptors will determine which mechanism predominates. McKinney and Conway (14) reported the calculated theoretical yield for different hydrogen acceptors for microbial systems utilizing acetate. The actual yield will probably be less because of the energy transfer efficiencies. The energy yields are shown in Table 2. It is obvious that if oxygen or nitrates are available, the predominate population will be aerobic organisms.

TABLE 2
Theoretical Energy Yields — Acetate Substrate

Hydrogen Acceptor	Energy Yield $\frac{\text{kg - cal.}}{\text{mole}}$
Oxygen	202.6
Nitrates	187.0
Sulfates	11.2
Carbon dioxide	7.0

Methane fermentation will not occur due to the adverse environment. More important in the anaerobic system is the difference between the energy yield from sulfate and carbon dioxide reduction. When sulfates are present, they will be reduced at the expense of carbon dioxide reduction. This results in the increased generation of the undesirable hydrogen sulfide with a corresponding decrease in methane production. The hydrogen sulfide may or may not be released from the liquid. This will depend upon the pH and presence of precipitating agents such as heavy metals. If the sulfide is released to the atmosphere, air pollution becomes a problem. If the sulfide remains in the liquid, it will exert an oxygen demand when discharged to an aerobic aqueous system. *Thiobacillus*, for example, is capable of oxidizing the sulfide to sulfate. Therefore, the presence of sulfate in the wastewater is likely to be a source of problems.

pH - ALKALINITY RELATIONSHIPS

The organic acids produced in the hydrolysis of the complex organics will cause the pH of the system to be depressed. The maintenance of the pH in an optimum range for the methane bacteria is important. The variation in pH is controlled by the buffer system developed during the decomposition of the waste material.

The carbon dioxide generated by decarboxylation of the organic compounds will react with water (Equation 2) to form carbonic acid. The

$$CO_2 + H_2O \rightleftarrows H_2CO_3 \qquad \text{.......(2)}$$

carbonic acid will ionize to produce bicarbonate ions and hydrogen ions. (Equation 3). These hydrogen ions depress the pH.

$$H_2CO_3 \rightleftarrows H^+ + HCO_3^- \qquad \text{.....(3)}$$

The quantity of carbonic acid formed will be a function of the partial pressure of carbon dioxide in the gas phase. This partial pressure is determined by the quantities of different gases produced as well as the ability of the system to retain the gasses. In conventional digestion tanks the gases are kept in contact with the digesting liquor. However, an anaerobic lagoon with little or no scum cover will lose most of the carbon dioxide. This can result in an increase in pH due to the loss of carbonic acid.

A second reaction occurring as a result of the deamination of the protein will tend to increase the alkalinity (bicarbonate) and the pH. The ammonia released from the protein is hydrolyzed to form ammonium hydroxide (Equation 4). Of course this hydroxide is capable of reacting

$$NH_3 + HOH \rightleftarrows NH_4^+ + OH^- \qquad \text{......(4)}$$

with the hydrogen ion generated by the ionization of carbonic acid. In simple terms, the effect of the production of ammonium hydroxide is the neutralization of the carbonic acid, producing buffer in the form of ammonium bicarbonate. McCarty (15) reported the relationship between bicarbonate alkalinity, pH and carbon dioxide content of the gas as shown in Figure 3. The relationship between the pH and bicarbonate alkalinity is strongly influenced by the carbon dioxide content of the gas in contact with the digesting liquor. In an anaerobic lagoon the partial pressure of the carbon dioxide could be substantially reduced by diffusion into the atmosphere. It would not be surprising to find pH values for a given bicarbonate alkalinity substantially higher in an anaerobic lagoon than in a conventional anaerobic digester. This also suggests that the anaerobic lagoon may have a lower buffer capacity, being more susceptible to pH changes.

The analytical technique for measuring alkalinity does not distinguish among the various forms. Any ion that is capable of reacting with hydrogen ions will register as alkalinity. This includes such ions as phosphates and sulfides. Also the volatile acids are capable of exhibiting alkalinity. At neutral pH values, the organic acids are ionized and are in the form of acetate, propionate etc. ions. When adding acid to determine the alkalinity, the hydrogen ions react with the salt ion forming the associated acid molecule. McCarty (15) gives the following reaction to depict the relationship between the "volatile acid" alkalinity and the bicarbonate alkalinity.

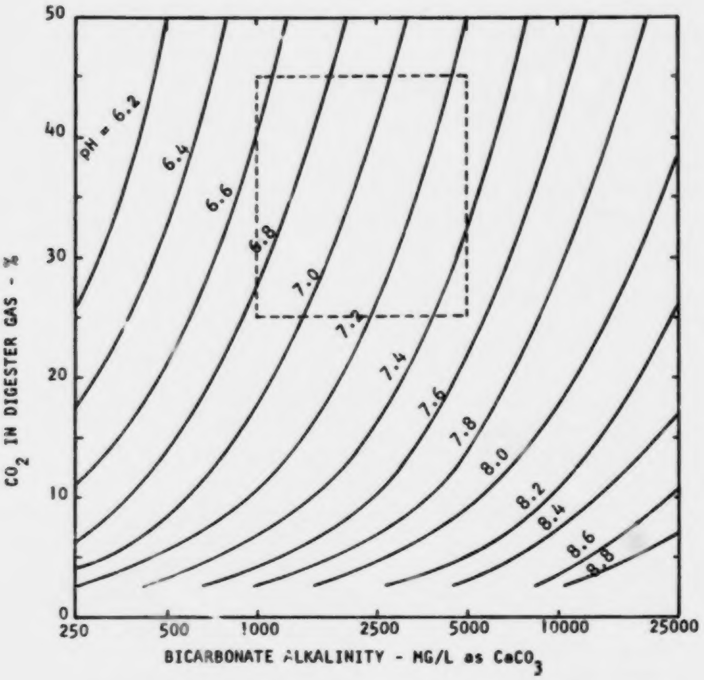

Figure 3. Relationship between pH and bicarbonate concentration near 95°F. After McCarty (15).

$$BA = TA - (0.85)(0.833)\,TVA \qquad \ldots\ldots(5)$$

where

BA = bicarbonate alkalinity, mg/l as $CaCO_3$
TA = total alkalinity, mg/l as $CaCO_3$
TVA = total volatile acid concentration, mg/l as acetic acid

This formula is similar to that developed by Pohland and Bloodgood (16), but includes a factor (0.85) to account for the fact that only 85 percent of the volatile acid alkalinity is measured by titration of total alkalinity to pH 4. This equation also assumes there is no significant concentration of other acid salts which produce alkalinity.

Anaerobic systems having a high measured total alkalinity can exhibit a low pH because the measured alkalinity includes the volatile acid alkalinity. If there is a significant amount of bicarbonate alkalinity, an acceptable pH will be maintained. This pH will be greater in the anaerobic lagoon than a conventional digester because of the loss of carbon dioxide to the atmosphere.

The methane bacteria are particularly sensitive to the pH level. The optimum pH range is 7.0 to 7.2. Methane fermentation rates are adequate as long as the pH is between 6.6 and 7.6. Severe retardation of the methane bacteria occurs if the pH drops below 6.6. Continued operation at pH levels below this minimum will completely inactivate methane fermentation. The acid forming microorganisms are not nearly as sensitive to pH level. Acid production will continue until the pH drops to about 4. At this pH, essentially all microbial action stops and what is commonly referred to as a "stuck" digester results. Maintaining the pH between 6.6 and 7.6 is important in achieving the desired operating efficiency from an anaerobic process.

TOXICITY PROBLEMS

Being a microbiological process, methane fermentation is subject to various toxic materials. Selective toxicity is commonly experienced in these systems because of the sensitivity of the methane bacteria. Several simple chemical ions have been observed to exhibit toxic effects on the methane fermentation. These effects can be attributed to the following five categories; alkali and alkaline-earth cations, heavy metal ions, ammonia, sulfide, and certain organic compounds.

McCarty (15) has summarized the effects of the alkali cations as shown in Table 3. Certain concentrations of these ions have been found to stimulate the methane fermentation. A continued increase in the concentration of these ions gradually inhibits the activity of the methane bacteria. These concentrations are normally found only in certain industrial wastewaters. Inhibition by a specific cation can be lessened by the addition of stimulatory concentration of another cation. However, this procedure is complex; requiring a thorough understanding of the

TABLE 3

Stimulatory and Inhibitory Concentration of Alkali and Alkaline-Earth cations*

	Concentration in mg/l		
Cation	Stimulatory	Moderate Inhibition	Strong Inhibition
Sodium	100-200	3500 5500	8,000
Potassium	200-400	2500-4500	12,000
Calcium	100 200	2500-4500	8,000
Magnesium	75-150	1000-1500	3,000

*After McCarty (15)

TABLE 4
Solubilities of Heavy Metal Salts - 25°C

Salt	K_{sp} - moles/liter
$Cr(OH)_3$	6×10^{-31}
Cu_2S	2.5×10^{-48}
PbS	7.1×10^{-29}
$Hg_2(OH)_2$	1×10^{-23}
HgS	1×10^{52}
NiS	1×10^{-24}
ZnS	1.6×10^{-23}

associated mechanisms. It should be approached with a great deal of caution.

Heavy metals, in particular copper, zinc and nickel, are known to retard the anaerobic process due to the metal toxicity. The concentrations that are toxic to a particular system is dependent upon the chemical composition of the waste stream. To be toxic, these metals must be in solution. These metals easily form insoluble salts by reacting with hydroxide, carbonate, sulfide or phosphate. These anions are common in anaerobic systems and in most cases metal toxicity is not a problem. However, waste streams containing high concentrations of the heavy metals or complexing agents can develop sufficiently high concentrations of soluble metals to cause toxicity.

The most important anion in the control of metal toxicity is sulfide. Metal sulfide salts have very low solubilities. Sulfates are common in most waters; which provides a source of sulfides. Additional sulfide can easily be added to the process for control of metal toxicity. Table 4 lists the solubility of the various salts of the heavy metals as given by Bard (17). From these solubility product constants, it can be seen that a relatively low soluble sulfide concentration will maintain a very low level of soluble metal ions.

Care must be exercised in adding sulfides to the process either directly, or in the form of sulfates. Sulfides exhibit a toxic effect on the methane fermentation process. The toxicity is directly related to the level of soluble sulfides. This level is influenced by the presence of metals, the pH and the partial pressure of hydrogen sulfide gas. The hydrogen sulfide ionizes according to Equation 6.

$$H_2S \rightleftharpoons H^+ + HS^- \qquad Ka = 1.1 \times 10^{-7} \quad \ldots\ldots(6a)$$

$$HS^- \rightleftharpoons H^+ + S^= \qquad Ka = 1 \times 10^{-14} \quad \ldots\ldots(6b)$$

The concentration of sulfide ions becomes a function of pH and the distribution of the ionic species is shown in Figure 4. Below pH of about 7.0, H_2S is the predominate form of sulfide. Because of the relative low solubility of hydrogen sulfide, the soluble sulfide will be low at this pH. As the pH increases, the soluble sulfide will increase due to the ionization of H_2S and a reduction of the concentration of H_2S gas in solution. Concentration of soluble sulfides varying from 50 to 150 mg/1 can be tolerated. Increases about this level causes inhibitions of the microorganisms. A concentration of 200 mg/1 or greater have been observed to be quite toxic (15). In an anaerobic lagoon, sulfide toxicity would be minimized because of the loss of the gas to the atmosphere. This of course causes odor problems that may require corrective action. In general, wastewaters high in sulfates or other forms of sulfur may be expected to cause problems in the anaerobic lagoon unless mechanisms for control of the sulfides are provided.

Another potential toxic component in the anaerobic system is ammonia nitrogen. Ammonia results from the deamination of the protein in the waste stream. As mentioned earlier, the production of ammonium hydroxide by the hydrolysis of ammonia is important in developing an adequate buffer system. However, wastewaters having a high organic nitrogen content can develop inhibitory conditions that have been traced to high concentrations of ammonia. This inhibitory effect is related to pH. At higher pH values, the effect is more severe. This is attributed to the presence of ammonia rather than ammonium ion. The relationship is shown in Figure 5. At a pH below 7.2, the ammonium ion accounts for essentially all of the ammonia nitrogen.

Ammonia nitrogen concentrations between 1500 and 3000 have been found to be toxic at a pH range of 7.2 to 7.4 (18, 19). This toxicity results from the presence of ammonia gas. Lowering the pH range to 7.0 to 7.2 can correct this toxic condition. Increasing ammonia nitrogen concentrations to levels of 3000 mg/1 has been found to be toxic regardless of pH. At this level, the ammonium ion also exhibits toxic effects.

An anaerobic lagoon could be very susceptible to ammonia toxicity when receiving a waste high in organic nitrogen. The loss of carbon dioxide to the atmosphere will elevate the pH. This pH elevation is aggravated by the production of ammonium hydroxide from the hydrolysis of ammonia. With the higher pH values, relatively low ammonia nitrogen concentrations can cause inhibition.

The possibility of toxic conditions resulting from certain organic compounds should be considered particularly with industrial wastewaters. Organic solvents, cleansing agents, sterilizing agents such as ethylene oxide, alcohols and long-chain fatty acids could be toxic to the

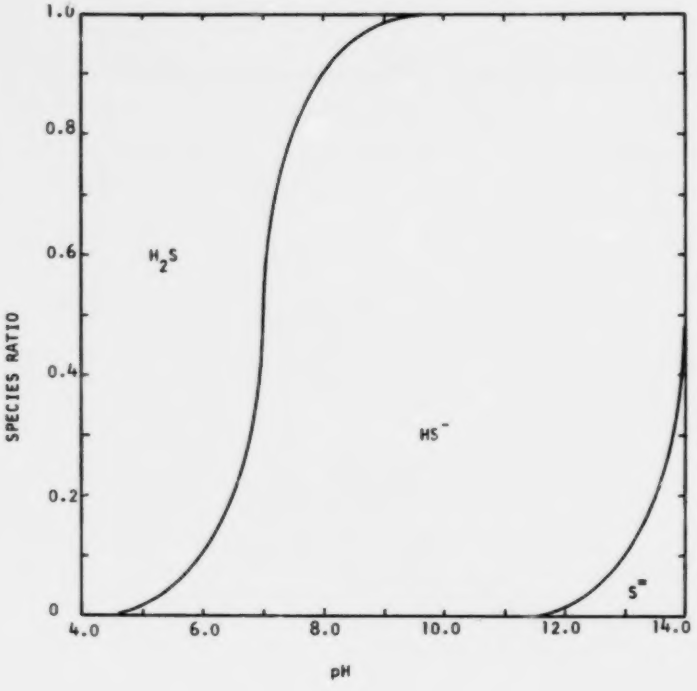

Figure 4. Distribution diagram for the hydrogen sulfide equilibrium system.

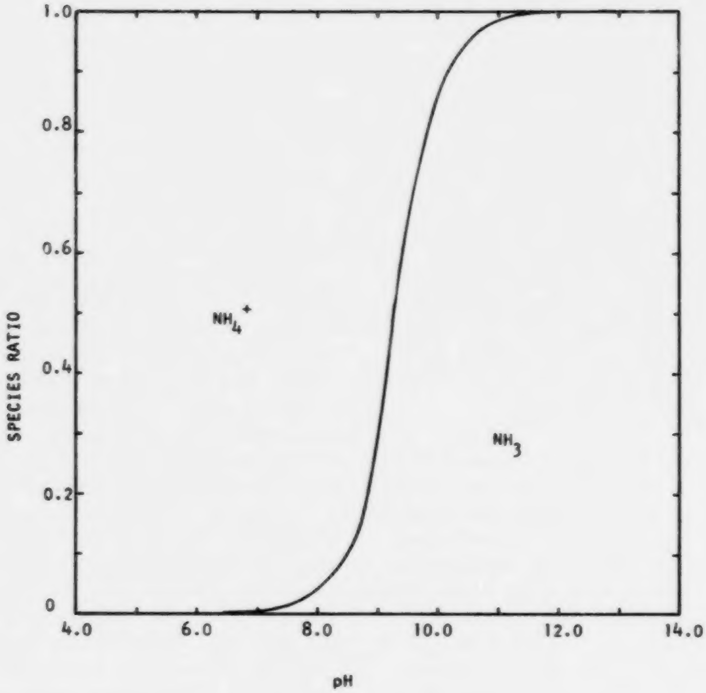

Figure 5. Distribution volatile solids destruction with solids retention time.

TABLE 5
Estimated Effect of Temperature on Rates of Anaerobic Treatment and (t_s) min*

Temperature (°C)	Relative Rate**	Relative (t_s)min**
5	0.1	10
15	0.4	2.5
25	0.8	1.2
35	1.0	1.0

*After McCarty (2)

**Relative to value at 35°C

TABLE 6
Effect of Temperature on Metabolic Rates in Aerobic Systems*

Temperature (°C)	Relative Rate**	Relative (t_s)min**
2.5	0.02	50.0
5.0	0.05	20.0
15.0	0.25	4.0
25.0	0.5	2.0
35.0	1.0	1.0

*After Cox (20)

**Relative to value at 35°C

microbial system. If these materials are discharged continuously at a relatively low rate, many of these organic toxins will be metabolized by the microorganisms with no inhibitory effect. However, slug discharges can cause complete upset of the system. Consideration must be given to control of the discharge of these materials to the lagoon system such that toxic conditions do not result.

WASTE STABILIZATION

The rate of stabilization of the organic material in the wastewater is a function of many factors. The most important of these factors related to the anaerobic lagoon are the temperature and retention time. Temperature control is not practical in these systems and temperature changes can be expected as a result of seasonal variations. Metabolism of the waste organic material is a biochemical process and, as such, is susceptible to temperature changes. Increasing temperatures result in increasing rates of waste stabilization until the temperature for thermal inactivation is reached.

McCarty (2) evaluated the results from earlier studies in municipal waste treatment and summarized them as shown in Table 5. These data indicate a marginal decrease in the reaction rates as temperature is reduced from 35° to 25°C. There was a sharper rate reduction in the next 10°C temperature drop. However, the decrease in reaction rate in going from 15° to 5°C was much larger. This may be correct, but information below 25°C is relatively limited and therefore positive conclusions cannot be drawn. However, work on aerobic processes (20) has indicated a similar relationship in the o° to 35°C range, as shown in Table 6. The temperature effect here is greater than that shown in Table 5.

Temperature becomes important in the rate of endogenous burnup or the microorganism decay rate. The effect of a decreased rate of sludge burnup is one of increasing the sludge production per unit of organic material. This will mean considerably more sludge to be disposed of ultimately or considerably longer solids retention time at lower temperatures to allow for maximum reduction in the quantity of sludge produced. There is very little information on the microorganism decay rate in anaerobic systems. McKinney (21) reports significant reduction in this decay rate with decreasing temperature in aerobic systems.

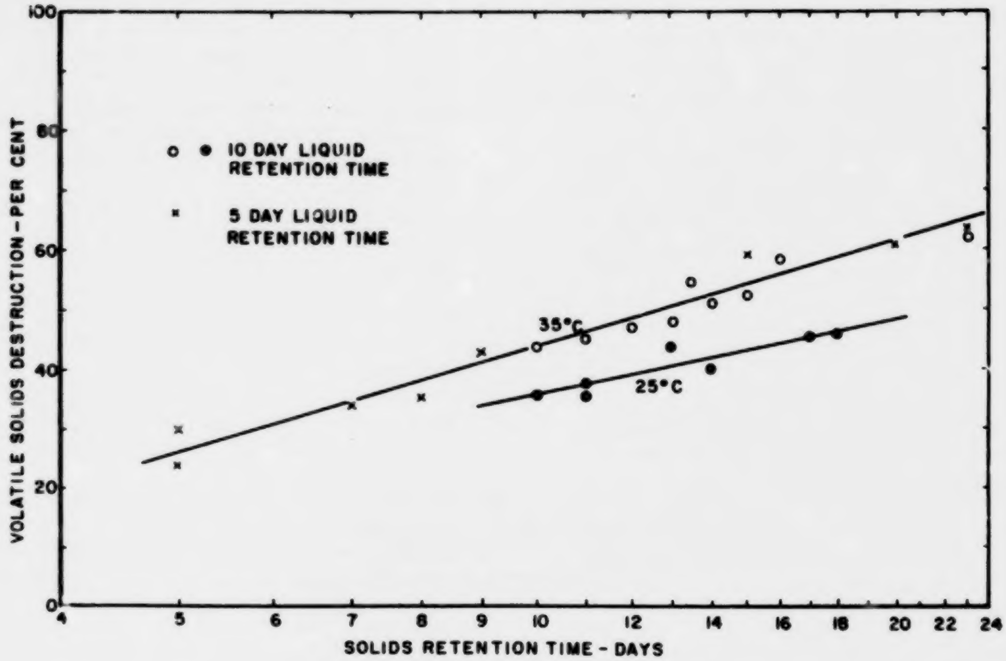

Figure 6. The variation in volatile solids destruction with solids retention time.

Therefore, it becomes apparent that temperature has a marked effect on the kinetic relationships in an anaerobic system. To compensate for the rate decrease with decreasing temperatures ample time must be provided to obtain the desired degree of waste stabilization. The important time factor is the solids retention time, not the liquid retention time. In any biological system employing recycle of biological solids, the solids retention time will determine the biological efficiency of the process. In an anaerobic lagoon, the solids retention time will be greater than the flow through time because the solids will settle. Since the lagoon system is not designed to control the solids retention time, the increase in efficiency due to this effect is difficult to ascertain. However, the lagoon should be designed to retain the solids. This provides a longer time for solids destruction and reduces the organic strength of the effluent.

The importance of the solids retention time in stabilization of organic solids is shown in Figure 6. The longer the time factor, the greater the distruction of the organic solids. These data were obtained from laboratory studies of the digestion of a mixture of primary plus waste activated sludge (5, 6). The digesters were operated at liquid retention times of 5 and 10 days. The solids retention time was increased by recycle of digested sludge from the secondary digesters. This figure also shows the effect of temperature on volatile solids destruction. Operation at 25°C required a 47 percent increase in the solids retention time to achieve the same degree of solids destruction.

SUMMARY

The anaerobic lagoon system has definite advantages in terms of costs over many other wastewater treatment systems. However, its application is not universal. There are many wastewaters that should not be treated by this system. The toxicity problems resulting from heavy metals, sulfides and ammonia are not easily controlled in this system. Also the odor associated with hydrogen sulfide release may be unacceptable.

The effect of temperature on methane fermentation must be considered. Excessively low operating temperatures will reduce the treatment efficiency or require excessive retention times for waste stabilization. Wastewater containing only soluble organics would require adequate liquid retention times to assure the required treatment. Organics in the form of suspended material will settle in the lagoon, greatly increasing the solids retention time. Because of the microbial populations that develop in the anaerobic system, many relatively stable materials, such as cellulose, will be digested.

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ANIMAL MANURE LAGOONS, A QUESTIONABLE TREATMENT SYSTEM

Samuel A. Hart

Once upon a long time ago, farmers thought that manure lagoons were the magic wand of waste disposal. A decade ago farmers believed that if they would only dig a hole in the sand and bury their animal manure in it, the good fairy godmother would make all their waste problems disappear. Manure lagoon success has not been quite that spectacular; although manure lagoons can be made to work well, they are not a universal answer to the manure problem.

The historical development of manure lagoons is interesting in itself, and furnishes a record of knowledge gained by agricultural engineers in the principles of sanitary engineering, and a record of progress in proper design of waste treatment facilities.

As indicated in the Proceedings of the First Lagoon Conference, Texas and North Dakota pioneered the development of sewage lagoons (1, 2). In 1953, the Missouri State Engineer visited sewage lagoon operations in North Dakota, and then began recommending lagoons for treating the sewage from rural towns of Missouri. True to the adage, once Missouri farmers had been shown the success of something, they were quick to adopt it. Consequently, manure lagoons began to be used for swine manure treatment. In the 1960 Lagoon Proceedings, Ricketts (3) pointed out that in 1959 about fifty manure lagoons were in operation, with twice that number planned for construction in 1960.

The records do not indicate the basis for design of these initial swine manure lagoons, although Ricketts recommended 15 ft^2 of surface area, 3 ft minimum depth, per head capacity of the swine confinement unit. Certainly this design was in no way related to the BOD_5 loading/acre typical of sewage lagoons. (As is discussed later, a 45 ft^3/animal loading is about 40 times the 1960 recommended loading for sewage lagoons.) In all probability, the manure lagoon design was based on how great an area the farmer could be talked into by the Farm Advisor, and 15 ft^2—approximately equal to the pen area per pig—was about as much as could be expected. Since hog operations are not perfume factories, the resulting septic, sour, malodourous lagoons were probably quite well masked by the normal pig odor.

Table 1. Water volume and loading rates for anaerobic manure lagoons

Livestock	Normal weight lbs*	Recommended lagoon volume, ft^3/lb animal	Manure solids Prod., lb/lb animal	Manure BOD_5 strength, lb/lb animal	Lagoon loading lb BOD_5/ft^3-day
Poultry	4	3	014	0.0040	0.0013
Swine	200	2	013	0.0033	0.0017
Cattle	1200	1	020	0.0012	0.0012

*The 4-lb chicken is an egg-laying hen, the 1200-lb cattle weight is that of a lactating Holstein dairy cow. Swine are raised for meat, and are usually marketed at 200 lbs. Fryer chickens, turkeys, and beef cattle are also raised, typical market weights for these are 3, 20, and 1000 lbs, respectively, and the manure strength data can be used to calculate the manure lagoon volume needed for such livestock.

With the so-called success of manure lagoons in Missouri in the late 1950's and early 1960's, the rural press began to tout them as the magic wand of wastes management. Lagoons were built by farmers from Maine to California, without any real scientific background upon which to base the design. However, researchers were concurrently beginning experimental studies on manure lagoons, and a number of publications appeared between 1963 and 1965 (4, 5, 6, 7, and 8 are representative). One result of this early scientific work was recognition that manure lagoons are more comparable to open-topped, unheated, unmixed sludge digesters (sludge lagoons) than to the typical sewage lagoon. Appreciation of this fact now seems very logical, because sewage lagoons are designed to upgrade the quality of a water relatively lightly diluted with organic matter, whereas manure lagoons, like sludge digesters, are designed to biodegrade and stabilize organic matter that is polluted with water. The similarity of manure slurries going into a lagoon, and sewage sludge into a digester, suggested using the terminology of digester loading rates for manure lagoons. The 1963-1965 publications suggested that satisfactory manure lagoon operation could be expected if the loading was approximately 0.01 lb VS/ft^3-day; this is just one-tenth the 0.10 lb VS/ft^3-day typical of heated, mixed municipal digesters, and the unsophisticated construction and operation of the manure lagoon is the argument for the lower loading rate. The important factor was recognition that the usual manure lagoon is an anaerobic digestion unit.

Anaerobic Manure Lagoons

The design criteria for anaerobic manure lagoons has advanced two steps beyond expressing loading rates expressed in lb VS/ft^3-day. First, consideration was given to designing lagoons on their ability to satisfy the BOD_5 Although this is logical in the sense that the ultimate objective of manure treatment is stabilization (BOD_5 reduction), the manure lagoon is not an aerobic vessel, and the wrong connotation is suggested. For the record, however, Hart (9) recalculated loading rates and suggested that manure lagoons could generally handle 1 to 1.5 lb BOD_5/1000 ft^3-day depending upon the location (climatic influence). Compared to the 1960 Lagoon Symposium recommendations for sewage lagoons (10), about 25 lb BOD_5/acre in a 5-ft deep lagoon, the anaerobic manure lagoon can handle 10 to 12 times the BOD_5/ft^3 that aerobic sewage lagoons can handle.

Today, the simplest, and most satisfactory, basis for design is to express the required lagoon volume in terms of the contributing livestock weight. The first three columns of Table 1 repeat the data from the current Agricultural Extension Service leaflet which advises farmers on lagoon design (11). The last two columns (calculated from data in Reference 12) indicate the manure strength (BOD_5/lb body weight), and lagoon loading in BOD_5/ft^3. Incidentally, this Agricultureal Extension Service leaflet is today's best presentation on how to actually design, construct, and operate manure lagoons.

As noted, the manure lagoon volume recommendations are based on the constant weight chicken and cow, and on the market weight pig. Thus, though the 200-lb pig overloads a 400 ft^3 lagoon during the last weeks of his life, he underloads it during much of his life. Lagoons have the ability to absorb such fluctuating loadings, as long as the average stays within the recommended limit.

Using the above volume: body weight factors, anaerobic manure lagoon design has come a long way since 1959 A few reservations remain, and can be discussed. The problem of malodors from lagoons is perhaps of most lay concern (13, 14). The original recommendations called for lagoons to be 3-5 ft deep. Since anaerobic manure lagoons do not benefit from free oxygen, there is no need for a large surface for oxygen transfer, photosynthesis, or odor production. The current recommendation for manure lagoons, therefore, is for depths of 8, 10, 12, or more feet, providing soil conditions are such that infiltration is prevented. Not only does such depth reduce the land requirement and surface that emanates odors, but it greatly assists in the uniform distribution of manure solids which form a sludge blanket on the bottom of the lagoon. The unacceptability of odors is the major drawback to lagoons in many areas. Perhaps in the truly rural areas, the odor problem is not serious, but in most parts of the country this one factor will significantly curtail lagoon construction.

There is also the question about infiltration from manure lagoons. Hart (8) found a ½-in/day infiltration rate even after two years of operation of pilot lagoons; the expected biological sealing of the soil did not occur Other researchers were evidently not plagued with this problem because no mention is made in other reports. If infiltration occurs, the percolate unquestionably pollutes subsurface waters. Although all states have laws on the books to prohibit such pollution, it is probable that some subsurface contamination still occurs. Most concerned researchers worry more about this long-range problem than about odors, efficiency, or anything else. Certainly prevention of subsurface contamination is a must if manure lagooning is to be a truly acceptable wastes management system. It is of course possible to seal the lagoon—with a lining of clay, plastic film, or soil cement, but the cost and complexity of doing this negates much of the attractiveness of the lagoon to the farmer.

If there is no infiltration, and evaporation does not remove sufficient water, the constant input of manure plus carrying water, and rain, will surely overfill the lagoon. Some farmers recycle the top liquor as the carrying water to bring the manure into the lagoon. However, most lagoons need to provide for some overflow. This effluent is often quite polluted, and may have a BOD_5 of 500-5000 mg/1. Research has been conducted on applying it to the soil as a sort of liquid fertilizer (15, 16). Treatment of this effluent is of course possible, but adds to the expense and complexity of the lagooning concept.

A manure lagoon is basically a processing unit, it is not generally the ultimate sink for the disposal of manure. Over a period of time (months to years, depending upon kind of manure, loading rate, and efficiency of organic material breakdown), the lagoon volume will gradually fill up with solids. Eventually, either the sludge has to be removed, or the lagoon abandoned and a new one dug. This is of course the same probelm that is faced in the operation of sludge digesters, but it is a fact not yet well-accepted by farmers. As the volume fills with degraded and non-reactive material, the volume/lb body weight of lagoon available to process incoming fresh manure is reduced, and a lagoon that operated properly in the beginning will become overloaded. As with any digester, the total contents should not be removed, but rather enough "seed" left in to assist biodegradation of the incoming waste. There are no definitive figures regarding volume consumption although two papers (17, 18) suggest that the bottom manure sludge runs 2-5% solids content; it can be calculated (and is borne out in practice) that manure sludge builds up at 0.02-0.1% of the lagoon volume per day.

Most of the incoming manure solids sink to the bottom and form the reactive sludge blanket. In the warmer months, the gas production occurring within this sludge may cause it to rise to the surface. The entrapped gas is then released, and the sludge again sinks. This "boiling" of the lagoon is not particularly aesthetic, but when it is occurring odors are usually minimal, and the lagoon is working its best.

Some incoming wastes do not naturally sink. Molted feathers from chickens are the worst offender. These build up into mats or rafts in the leeward corners of a lagoon, and become a site for flies to lay eggs upon. The farmer really should rake such floating matter off his lagoon—the sewage plant operator skims grease and similar floating matter off his primary tanks. However, the farmer's primary concern is managing his livestock, not managing their manure, and it is not likely that he will tend the lagoon to the degree that would be desirable.

Dairy and beef manure contains large quantities of hay stems and grain hulls, both of which tend to float. Thus the cattle manure lagoon often builds up a floating crust on the surface. In the original research on lagoons, this was thought to be undesirable. In retrospect, and in current practice, this crust is considered very desirable. The crust is composed of "washed" wastes, it is innocuous and non-supportive of fly larvae. As a cover for the lagoon, it reduces odors, and is no more unaesthetic than a water-surface lagoon. One ought not to try to walk on the crust, however.

Although the volume required per lb body weight seems quite small and reasonable, the lagoon size required for today's typical farming operation is still large. A 50-sow swine operation produces some 400 market pigs per farrowing. The lagoon thus needs to be designed for 450 swine, at 200 lb each, and 2 ft^3/lb body weight, a volume of 180,000 ft^3. A ½-acre lagoon, 8.2 ft deep would be needed. Including the side slopes and berm, nearly one acre of land would actually be used up.

In order to reduce lagoon size, or to fit today livestock numbers to yesterday's designed lagoon, farmers often use the lagoon for only a part of the total amount of manure produced. If the farmer has cropland upon which he can spread manure, he often will do that in summer and fall, reserving the lagoon for winter when he cannot get onto the fields. Alternatively, he may manage some of his livestock on litter or on pasture, and thus not need to use the lagoon for the wastes from these animals. Both such practices are very desirable; the consensus of numerous researchers, extension personnel, and practicing farmers is that lagoons should be the "reserve" method of manure management. An anaerobic lagoon is a good thing to be able to fall back to (figuratively, though not literally). When overloaded, it is malodorous and disagreeable; when properly designed and used, it can be satisfactory.

Aerated Manure Lagoons

In the same way that sewage lagoons served as the model for anaerobic manure lagoons, the floating aerators for sewage lagoons have been tested and are being used on manure lagoons.

There are two entirely different ways in which the floating aerator can be used in a manure lagoon. It can be used as a stirring and aerating device to give a completely mixed, aerobic liquor, similar to the aerated sewage lagoon. The second concept is to oxygenate only the top surface of a deep, anaerobic lagoon.

Dale et al (19) have reported on research where the aerators are used in the first manner, to make the lagoon into a mixed, aerobic reaction vessel. The lagoon is basically an activated sludge treatment facility. However, instead of clarifying the effluent liquor, it is used directly for sprinkler irrigation. Using a 1 ft^3/lb body weight lagoon for dairy manure (basically the same volume requirement as for an anaerobic lagoon), with an aerator oxygenation capacity equivalent to the daily BOD_5 demand, the lagoon operated quite satisfactorily. The aerator was sized at 0.10 HP/cow; the aerator oxygenates at abut 3.0 lb O_2/HP-hr, and the cow defecated 7 lb BOD_5/day. The mixed liquor solids was kept between 2 and 3 percent so that sprinkler irrigation of the effluent could be accomplished. This required the incoming manure to be diluted with water. Detention time is not reaily specifiable because during winter there cannot be (or should not be) any effluent. (It would be inappropriate to spread effluent on frozen ground because it would wash downstream during the Spring thaw). In summer, irrigation with the lagoon liquor may occur as a function of water need rather than of manure stabilization. Because of the winter sequestering of manure in the aerated lagoon, it must be partially emptied as Fall approaches. The advantages of the aerated lagoon is that it can take the total manure from the animals (no diverting necessary), and rainfall falling on the corral areas is beneficially used, rather than flowing off the farm to pollute downstream waters.

Comprehensive field studies with this mixed liquor principle of the aerated lagoon have not yet been conducted with other manures. Vickers and Genetelli (20) have conducted laboratory experiments on poultry manure. Their results indicate that 60 ft^3/ of aerated lagoon volume should be allocated per lb incoming daily BOD_5 (0.0167 lb BOD_5/ft^3-day). This is about ten times the loading rate suggested in Table 1 for anaerobic lagoons. Detention time, flow-through rate, and quantity of influent water are dependent upon the solids content of the mixed liquor, a break-point in the efficiency of the reaction occurred with a mixed liquor solids content of slightly less than 2 percent (19,660 mg TSS/l). Considerable additional research needs to be done with aerated lagoons, but this concept may be quite acceptable in the future. Since this method of aerated lagoon operation is based on flow-through, it will be necessary to have a wastewater disposal system available.

The second concept for an aerated lagoon is even less worked out, but has been cursorily evaluated. The original facultative sewage lagoon was conceived of as being anaerobic on the bottom, with algae-laden aerobic water at the surface. Foul gases rising from the anaerobic bottom would be oxidized—physically and/or biologically—as they rose through the aerobic top volume. It should be possible to duplicate this principle mechanically, with surface aerators oxygenating only the top liquor of the lagoon, and not disturbing the anaerobic depths. To date, there is no published literature on this system of manure lagooning, but research is under way at Purdue University. One can conjecture that the volume of lagoon necessary would be less than for the completely anaerobic lagoon (though larger than for a completely mixed lagoon), and the effluent problems would be minimal.

Oxidation Ditches

In 1960, Dr. A. Pasveer of Holland gave some lectures to the Manhattan College Conference on Biological Waste Treatment, on new concepts of sewage treatment (21). The new concept was the oxidation ditch, a shallow (less than 2 ft deep) race-track shaped trough with a paddle wheel. The paddle wheel circulated the water to keep the activated sludge floc suspended, and beat or churned oxygen into the liquor. Subsequently, detailed results of the system were published (22). Agricultural engineers had begun studying and learning sanitary engineering, so immediately began researching the potential of the oxidation ditch for manure treatment. Irgens and Day (23) conducted a laboratory study, including tests of an "in-house" ditch under the slatted floor of a swine house. Additional research (24, 25, and 26 are representative) have resulted in the publication of an Agricultural Extension Leaflet (27) which includes facts and recommendations for design, construction, and operation of "in-house" oxidation lagoons. Although the oxidation ditch principle would work equally well out of doors, there are distinct advantages for constructing the waste treatment facility as an integral part of the animal housing unit. Specific among these advantages is the elimination of the manure collecting and transporting chores, continuous rather than slug loading of the ditch, minimization of odors, both from the treatment facility and from the livestock (when the unit operates well), and reduced cost compared to conventional housing and an outside waste treatment unit.

To date, the research and the recommendations for farm use of the oxidation ditch have been restricted to hogs, but there is no reason why they could not also be used to treat poultry or cattle manure. Present design calls for 12 ft^3/200-lb pig, very much less than the 400 or 200 ft^3/pig required for anaerobic and aerated lagoons. Because the total urine and feces volume of a 200-lb swine amounts to about ¼-ft^3/day, an oxidation ditch would soon overflow if evaporation, degradation, and sludge removal did not occur. All do occur, including the design of a 2%/day (15% week) mixed solids removal. (This 2%/day mixed liquor removal is based on a 50-day detention time, and the removal of the material prevents buildup of salts, activated sludge floc, and aerobically digested sludge. Even with this effluent, the oxidation ditch liquor may run from 3 to 5% solids content. Because of the effluent and evaporation, some makeup (dilution) water is generally needed, and may come from spillage of drinking water, and from normal washing operations.

The mixed liquor typically has a BOD_5 of 3000-5000 mg/1, so it cannot be directly discharged. It can be applied to cropland without odor and with minimal pollution potential, or it can be treated (or stored) in a secondary faculative lagoon. In applying oxidation ditch design to poultry and cattle manure, the basic design criteria allots 30 ft^3/lb BOD_5-day of influent (0.033 lb BOD_5/ft^3-day), 50-day detention, and oxygenation capacity equal to twice the daily BOD_5 input.

Rotors to circulate and oxygenate the mixed liquor are commercially available. Their size, speed, depth of immersion, and horsepower are predicated on the volume of waste to be moved, and the amount of oxygen to be supplied. It is necessary to keep the liquor moving at about 1¼ ft/sec in the raceway to prevent sludge bank buildups. The rotor also churns in oxygen, the design being to supply twice as much oxygen as is needed to satisfy the BOD_5 of the fresh manure being accepted per day. Design data for specific rotors is available from individual manufacturers, typically 0.02 HP will be required/market pig. The rotor and motor assembly typically costs about \$300/HP, or \$6.00/hog. The cost of the raceway—the oxidation ditch proper—is additional. Operational costs for power run about 0.6¢/hog-day, at typical power rates.

Some problems with in-house oxidation ditches have occurred. Foaming of the slurry has been one problem. Often it can be controlled by increasing the aeration rate (speeding up the rotor and perhaps decreasing the depth of immersion), or by pouring oil upon the troubled water. Insufficient oxygenation, poor circulation, or overloading result in inadequate treatment and malodors. Electrical outages or mechanical failures also occur, and it is important to provide very good ventilation throughout the barn when restarting an oxidation ditch that has been out of operation for a day or more.

Conclusions

Manure lagoons (and oxidation ditches) are here to stay. As a treatment (and storage) facility for animal manures, they have a good deal to offer the farmer. Like any solution to any problem, they must be well-engineered and then be well-operated and well-maintained. Farmers are sometimes reluctant to spend the money and time necessary to obtain good performance. Lagoons and oxidation ditches are not magic wands, but they can be very reasonable manure processing units.

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OPERATIONAL EXPERIENCES WITH SLUDGE LAGOONS

(Tertiary Digesters)

by

C.H. Williams
Waste Water Treatment Engineer
City of Dallas Water Utilities

HISTORY

History of lagooning for treatment and disposal of sludge in the City of Dallas goes back to January 6, 1940. Until that time Dallas had only Imhoff tank treatment of sewage, and sludge was dried on more or less conventional drying beds.

In January 1940, a plant expansion and improvement project was completed and put into operation. Plant then consisted of grit channels, bar screens, primary settling and Imhoff tanks, trickling filters, and secondary settling basins; separate sludge digestion was provided in a two stage process. Final disposal of sludge from Imhoff tanks and digesters was provided by drawing sludge to a "sludge lagoon". This lagoon was about five feet deep and underflow from first lagoon flowed, in series, through six additional lagoons. (See "E" Figure II)

Waste Water Treatment facilities were again expanded and operations started in January 1953. An additional "lagoon" was built to treat and dispose of sludges produced from expanded treatment. This lagoon averaged about nine feet in depth. (See "D" Figure II) It was found that, in addition to digested sludge it was also necessary to use this lagoon for disposal of grit from a girt washer, which does not function well.

These "sludge lagoons, D and E" remain in service today. They are in the flood plain of the Trinity River, and although thirty years of data indicates effluent adds only about one ppm BOD to total plant effluent; they will be abandoned as soon as other facilities can be completed. This is one of those occasions when one must recognize THE-L law, "The theory is right, to the hell with the facts".

At various times in the past years the river "had cut through existing lagoons". There is considerable evidence that, contrary to popular belief in our area, "washing out" may have actually reduced treatment capacity of these units, rather than increased capacity or extended "life". In fact from some crude soundings, we have reason to believe more sediment was collected from river water than "ash" from biological "trash". Scum disipation is effective but of small value. For example; during drouth of the mid 50's the river did not "cut through" these units for a five and one-half year period. As indicated above, rather crude soundings and maintenance records indicate less "build-up" during this period than any other period, of record.

In the mid 1950's, the City of Dallas was planning a third major increase to plant capacity. At that time it was planned to do something "better" for solid disposals. We and our consultant traveled a good deal, collected a lot of data and talked to a large number of operators, engineers and administrators. All of this was done in an attempt to survey the then available processes for final disposal of sewage solids and sludges.

A report of various accounting methods and procedures discovered would; although out of date, make a very interesting paper itself. By taking a generous poetic license with data collected, we came up with adjusted and estimated local cost of a possible $20,000,000 capital investment to dry and/or incenerate one hundred tons per day of digested sludge. In addition to capital cost, a local evaluated operation cost was $30 per ton. Further, when

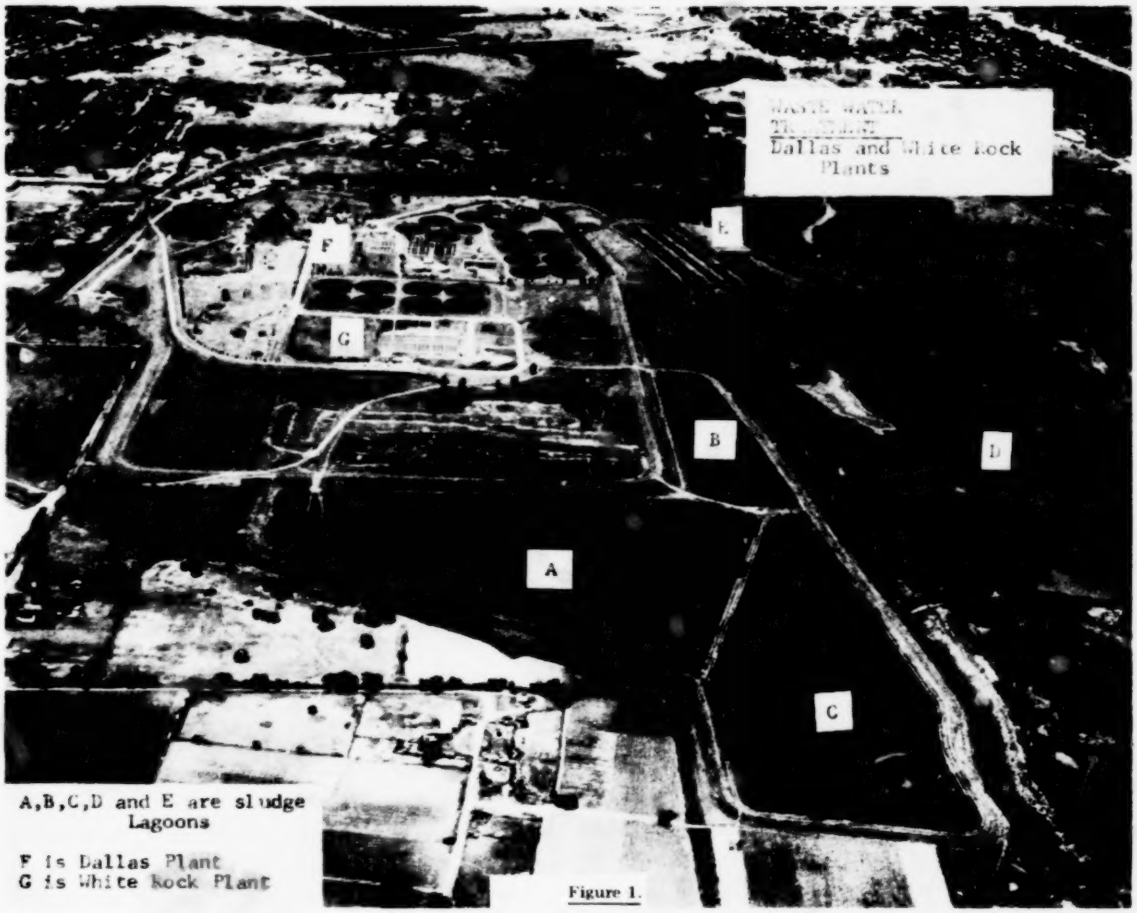

Figure 1.

we went to manufacturers we found that meaningful performance guarantees were out of the question. In general you paid your money and took your chances.

About this time we backed up and took a general look at what we were attempting to do. It was decided rather than "monkey with" processes which were very expensive and about which it was hard to make reasonably close predictions. We would pursue a process with which we had experience.

TERTIARY DIGESTERS

It was decided, based on the above, and the fact that calculations indicated only minor increased BOD in effluents, to build a more glorified version of these units. This brought about design and eventual placing in operation of the "Tertiary Digesters of the City of Dallas". (See "A", "B", and "C" of Figure 1)

These consist of an area which is divided into three separate lagoons:

Lagoon "A", includes a borrow area created when plant levees were built.(7' average depth)

Lagoon "B", includes areas from which gravel had been removed. (10' average depth)

Lagoon "C", was an excavated area graded and so constructed as to be operated between 3' and 5' depth.

At intersection of the three lagoons there is a flow control gate and pump well structure. This structure is so arranged that a 15 M.G.D. pump can take flow from any of the three lagoons. These flows can be taken from two outlet structures in "A" and one in each of "B" and "C" lagoons. Each outlet provides a sluice gate at normal bottom elevation of the lagoons. Above the sill four corner posts provide support for a double wood baffle and stop plank arrangement, on each of the four sides. The double plank arrangement permits selected withdrawal from surface or any other desired depth. At the same time a constant liquid level can be maintained. Sections of 1" steel grates with 1" clear openings are used as insert screens at the selected draw off level. There are two additional draw offs in outlet structure of "C" lagoon. These are two ten inch telescoping valves and allow diversion of effluent from "C" lagoon to plant outfall and/or to head of the White Rock Waste Water Treatment Plant.

Discharge rate from the 15 M.G.D. pump can be varied with a magnetic coupling and is made into a header with valves, for alternate points and various rates of discharge. From the pump a 30" discharge line is extended to the west end of lagoon "A", and a 24" branch to the north end of lagoon "B". Valves are provided in each branch. An alternate 36" discharge is provided with control valves, at

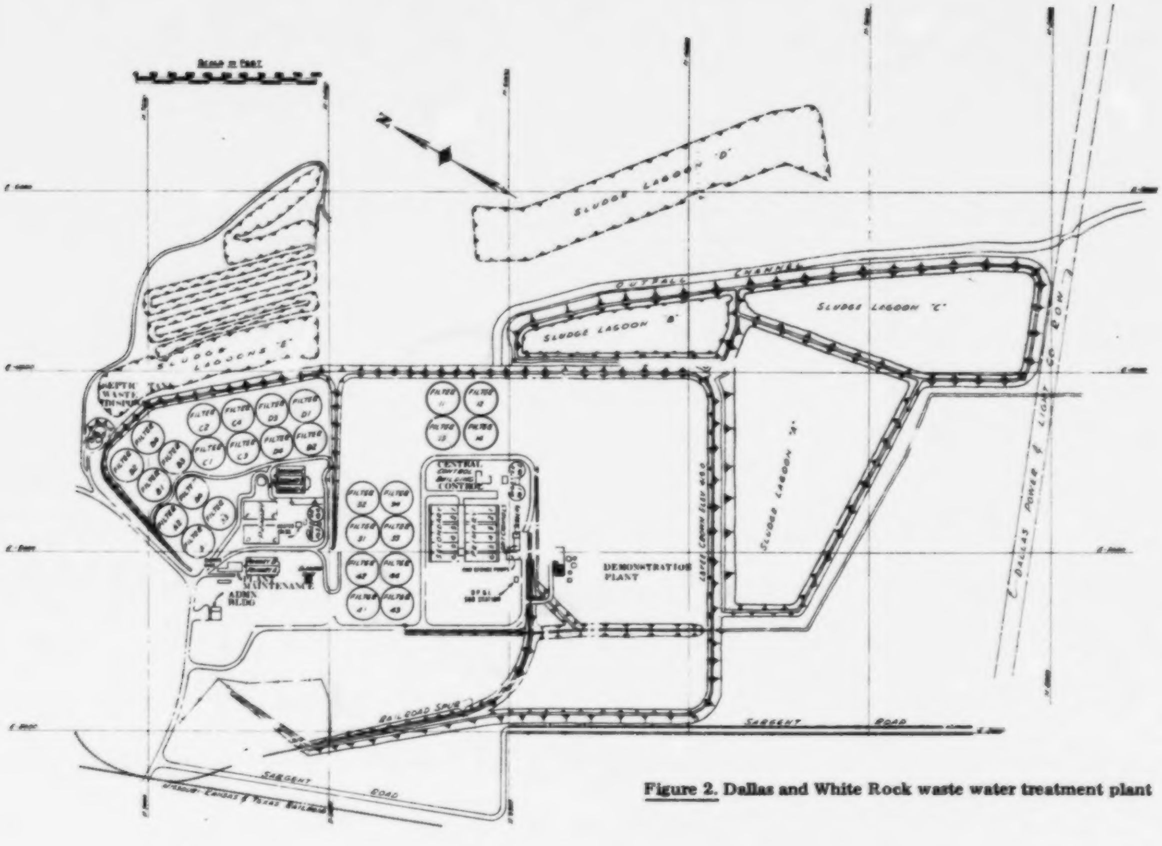
N
SLUDGE LAGOON 'D'
SLUDGE LAGOONS 'E'
OUTFALL CHANNEL
SLUDGE LAGOON 'B'
SLUDGE LAGOON 'C'
SLUDGE LAGOON 'A'
SEPTIC WASTE DISPOSAL
FILTER C2
FILTER C4
FILTER D3
FILTER D1
FILTER C1
FILTER C3
FILTER D4
FILTER D2
FILTER 11
FILTER 12
FILTER 13
FILTER 14
FILTER 32
FILTER 34
FILTER 31
FILTER 33
FILTER 42
FILTER 44
FILTER 41
FILTER 43
CENTRAL CONTROL BUILDING
SECONDARY
PRIMARY
DEMONSTRATION PLANT
PLANT MAINTENANCE
ADMN BLDG
LEVEE CROWN ELEV 420.0
DALLAS POWER & LIGHT CO. ROW
RAILROAD SPUR
SARGENT ROAD
SARGENT ROAD
MISSOURI-KANSAS & TEXAS RAILROAD

Figure 2. Dallas and White Rock waste water treatment plant

the structure, for pumping flows to lagoon "C". (See Figures III and IV)

Sludge can be fed to these lagoons by two 12" lines from the Waste Water Treatment Plant complex. These terminate in Tee connections to recirculation pump discharge lines. This allows feed either directly to any of the three lagoons or mixed with recirculated flow. Further, Lagoons "A, B, and C" were levied above the twenty-five year flood level of the Trinity River, and have level protection at operation level water line. This protection consists of eight feet long, heavy asphalt plates. Installation was made by overlapping plates and sealing joints. Ends are turned under one foot, giving a six foot exposed length and a three foot vertical protection.

OPERATIONS

Process was intended to be a "wet process" and since detention times (See Tables I, II, and III) are long; operations were initiated by filling lagoons "A, B, and C" with Waste Water Plant effluent. This allowed starting of recirculation pump and a general "shake down" of the installation. (See Figure III)

Several initial combinations were tried, but soon developed into "normal operation" of: Digested sludge feed from Waste Water Treatment complex is mixed with recirculation flow taken from outfall of "B" and discharged into inlet of "B" lagoon. Displacement caused by sludge feed is diverted to "A" inlet, by "cracking" valve on recirculation pump leader. This allows preferential discharge to "B" but keeps liquid operation level of "B" constant. Overflow from "B" becomes feed to "A"; effluent from "A" overflows by gravity; and constitutes feed into "C" lagoon. Any effluent which might be discharged from "C" is recycled to head of the conventional Waste Water Treatment Plant: Figure I; Shows units in normal operation; Figure, IV; indicates general operational layout.

Results from seven years operation, are summarized in Tables; IV and V. One word of caution, this is basically a series operation, "B" being a first stage, "A" a secondary stage and "C" a final or third stage.

Since everyone seems interested in Phosphates, Tables VI and VII are presented. We have not made a great many phosphate determinations, as tables indicate, but we probably will run many more in the future. During a period when recirculation pump was out of service, temperature was comparatively low, and cloud cover was almost continuous, disolved oxygen in "C" lagoon became depleted. This occurred January thru April 1969. The volume of "C" effluent pump No. "92", (See Figure V) gradually diminished during this same period.

Pump No. "92" is used to pump "C" lagoon effluent through "scum control" distribution piping. To make a long story short, we found the pump and impeller to be filled with a crystaline phosphate. Pump was cleaned and system is gradually dissipating phosphate, which had caked in piping.

TYPICAL SECTION-LEVEES BETWEEN LAGOONS

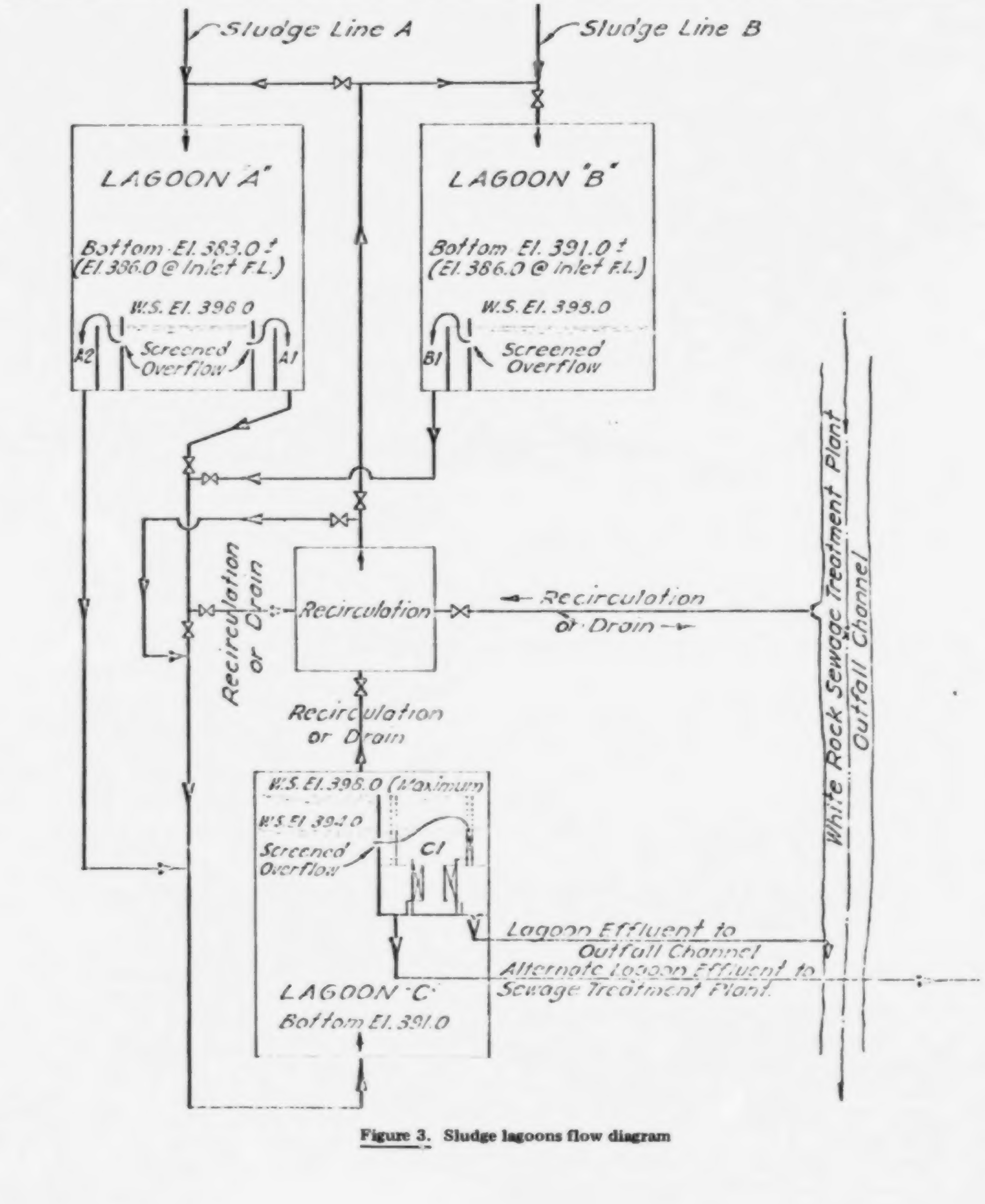
Sludge Line A
Sludge Line B
LAGOON "A"
Bottom El. 383.0 ±
(El. 386.0 @ Inlet F.L.)
W.S. El. 396.0
A2
A1
Screened Overflow
LAGOON "B"
Bottom El. 391.0 ±
(El. 386.0 @ Inlet F.L.)
W.S. El. 398.0
B1
Screened Overflow
Recirculation
Recirculation or Drain
Recirculation or Drain
Recirculation or Drain
W.S. El. 398.0 (Maximum
W.S. El. 394.0
Screened Overflow
C1
Lagoon Effluent to Outfall Channel
Alternate Lagoon Effluent to Sewage Treatment Plant
LAGOON "C"
Bottom El. 391.0
White Rock Sewage Treatment Plant
Outfall Channel

Figure 3. Sludge lagoons flow diagram

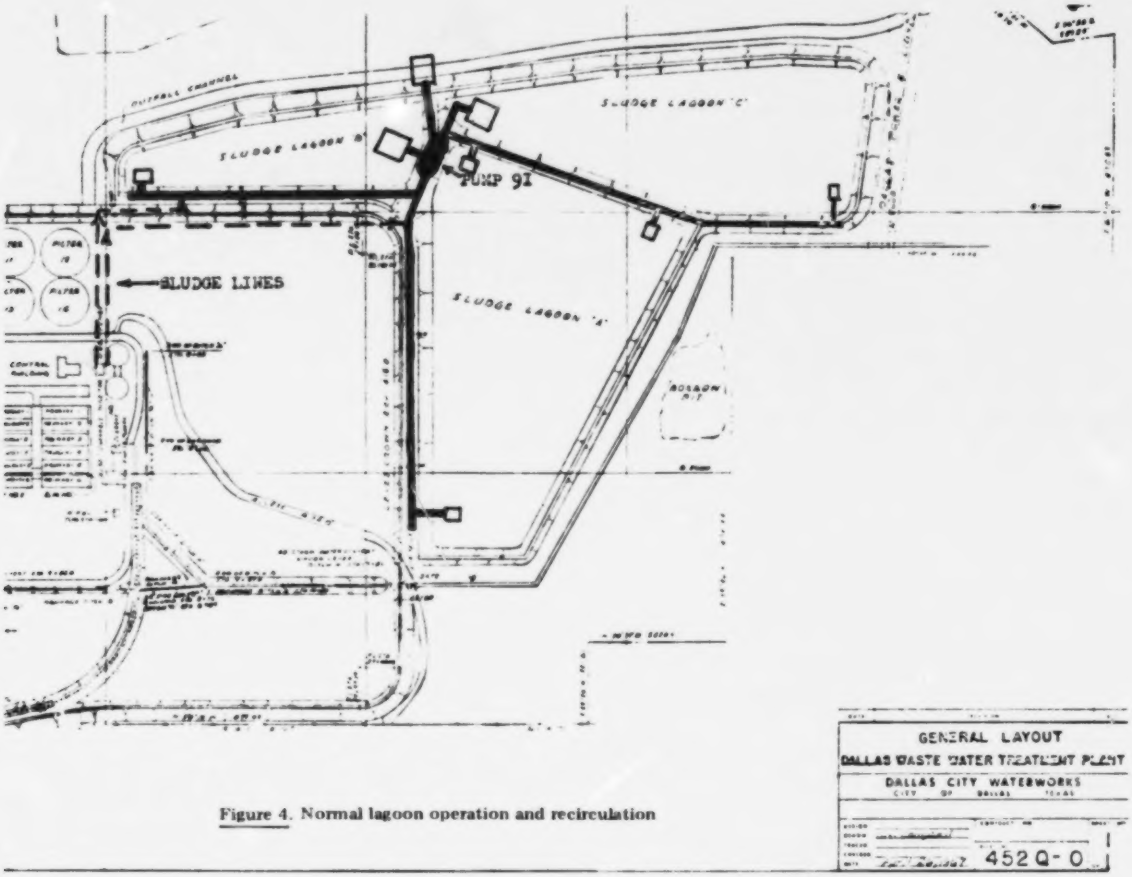
OUTFALL CHANNEL
SLUDGE LAGOON 'C'
SLUDGE LAGOON 'B'
PUMP 9I
SLUDGE LINES
SLUDGE LAGOON 'A'
BORROW PIT
GENERAL LAYOUT
DALLAS WASTE WATER TREATMENT PLANT
DALLAS CITY WATERWORKS
CITY OF DALLAS TEXAS
452Q-0

Figure 4. Normal lagoon operation and recirculation

Table 1. Tertiary digesters

Lagoon	A	B	C	D	E	Total
Area (acres)	29	7	16	16	11	79
Avg. Depth	7	10	3	9	5	--
Cap. Millions of ft.^{3}	9	3	2	6	2	22
Cap. (acre/ft.)	207	69	46	144	55	521
Date (days)	280	90	60	--	--	430
Year in Service	1963	1963	1963	1953	1940	

Table 2. Quantity of sludge from sewage

	Per/MG	from 100 MGD Sewage Flow
Suspended Solids Removed (Avg.)	2,000 lbs.	100 Tons
At 6% solids to formal digesters	3,500 gals.	350,000 Gals.
Digested, air dried 60% water, 35 lb/yd.^{3}	—	376 yd.^{3}
INCINERATED, ash 30 lb/yd.^{3}	—	71 yd.^{3}

Table 3. Expected useful life

Assume "bottom mud" to be:	Rate of accumulation ft^3/day	Time to fill Lagoons A & B (years)
Equivalent to air dried 60% water, 35 $lb/ft.^3$	10,000	3
Equivalent to air dried consolidated to 100 $lb/ft.^3$	3,500	9
Equivalent to "fired ash". 35 $lb/ft.^3$	1,930	16
Equivalent to "fired ash". 100 $lb/ft.^3$	675	52

Table 4. Operational results mg/l

Effluent of lagoons	A^{xx}		B^{x}		C^{xxx}		D^{1}		E	
Year	SS	BOD	SS	BOD	SS	BOD	SS	BOD	SS	BOD
1963	← Feed but no effluent →						264	320	730	180
1964	313	130	2,330	410	373	150	567	610	357	150
1965	475	180	4,500	600	383	140	176	150	99	130
1966	441	160	2,920	460	458	140	83	58	61	35
1967	491	200	4,650	810	407	140	217	150	79	53
1968	716	350	3,700	980	432	250	316	160	94	63
1969	679	300	6,446	970	413	170	2,060	330	110	270
Average	519	220	4,080	705	407	165	578	254	219	126

x B first stage

xx A second stage

xxx C third stage

[1] includes grit washer effluent

Table 5. A three month study of lagoons A, B, C average results June thru Aug. 1965

	Total Solids lbs.	pH	Suspended Solids mg/1	B.O.D. mg/1	% Volatile S.S.	O_2 Consumed mg/1
B-inlet	5,663	7.1	5,120	800	60.9	4,302
B-outlet 1 ft. depth	5,692	7.1	4,595	669	59.5	3,216
B-outlet 5 ft. depth	32,587	7.1	30,000	2,185	56.3	17,863
A-inlet	4,968	7.1	3,648	573	61.0	2,433
A-outlet 1 ft. depth	1,624	7.7	536	189	58.5	570
A-outlet 3 ft. depth	1,641	7.7	596	192	59.4	571
A-outlet 7 ft. depth	1,655	7.8	570	188	58.4	558
C-inlet	1,609	7.7	504	105	60.6	519
C-outlet 1 ft. depth	1,609	7.8	486	143	59.8	538
C-outlet 4 ft. depth	1,625	7.8	550	133	60.0	547

Table 6. Operational results mg/1 Phosphates in lagoon effluents

	(A)			(B)			(C)			(E)		
Year	Total	Ortho	Poly	Total	Ortho	Poly	Total	Ortho	Poly	Total	Ortho	Poly
1964	ND	50	ND	ND	ND	ND	ND	65	ND	ND	30	ND
1969	197	195	4	605	335	270	160	164	0	ND	ND	ND
1970	106	92	14	278	191	87	77	65	12	14	12	2

Table 7. Additional analysis during pump trouble of Feb. to April 1969

Lagoon	A	B	C
Total PO_4 mg/1	197	605	160
Ortho PO_4 mg/1	195	335	164
Poly PO_4 mg/1	4	270	0
Calcium mg/1	94	114	84
Magnesium mg/1	26	39	20

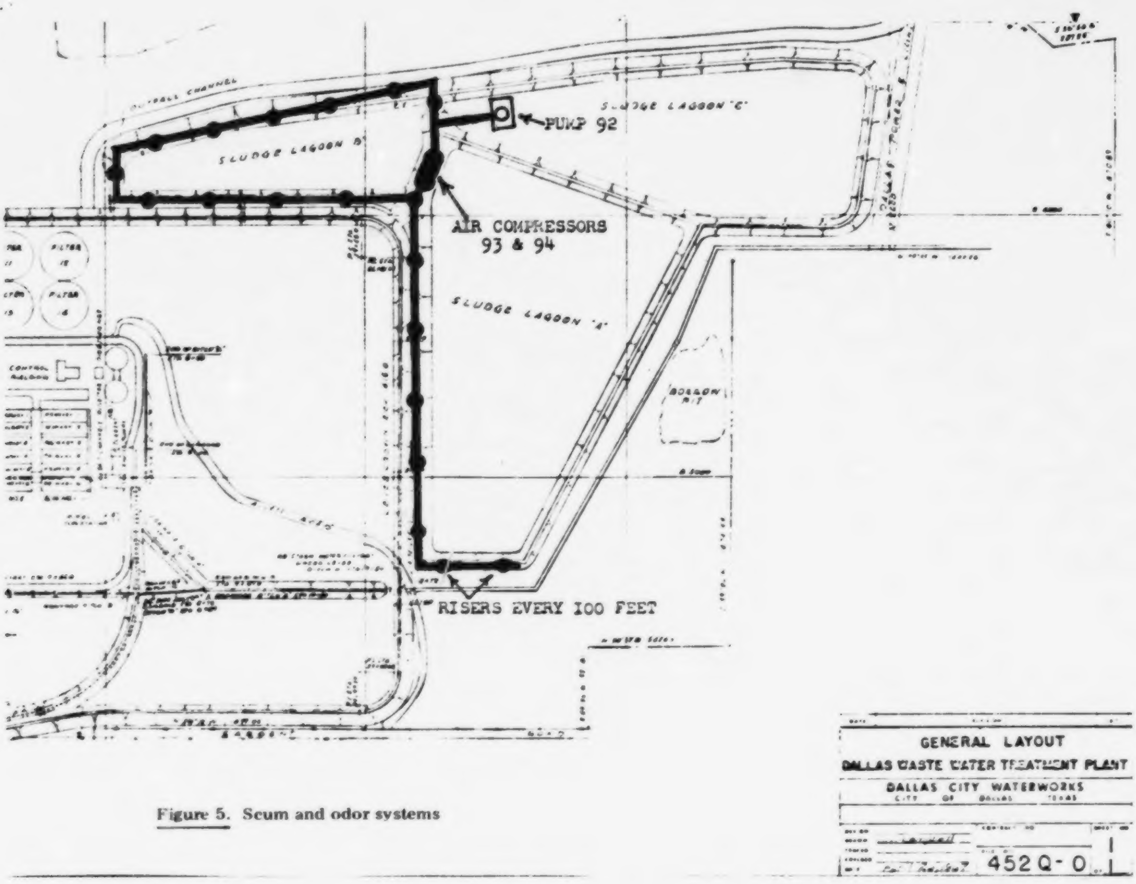

Figure 5. Scum and odor systems

Of course all operations have not been "sweetness and light", scum control and even odor control have been problems at times. We have found that by using effluent recirculated from "C" lagoon to "wet down and break-up" scum, generally it can be kept under control. In addition, our people have improvised a "barge-plow". By pulling this plow from the bank, with a cable hooked to a standard one yard dump truck, they have developed a technique which submerges scum and stirs lagoons rapidly and well. We have found that as long as we divert only "well digested" sludge to this system, we have had no odor complaints from the public, nor have we noticed any objectional odors. On the few occasions when odors have been strong, these odors have always occurred just after diversion of raw sludge to the system.

Odors are of particular importance to us since plants are completely surrounded by the City of Dallas. As a safety measure, in addition to effluent pump No. "92", two air compressors have been installed and their discharges connected to effluent water piping, (See Pumps No. "93" and No. "94", Figure V.) It is possible, if strong odors occur, to drain water from effluent piping system and install spray heads every 100 feet for odor control chemical dispersion. Air from these compressors at times has also been used to break up scum, dissipate sludge banks, etc.

SUMMARY

After a rather extended experience with "wet sludge" lagoons, from 1940 to 1960, a system of lagoons was constructed and put in operation in early 1963. These units were constructed and put into operation at a cost of $251,000 and have required about 100 man days per year operation. Electrical power costs are small, about $1,200 per year. This compares with and estimated $20,000,000 capital and estimated $1,000,000 per year operational cost, for a more conventional system.

Corings and soundings indicate about one third of operational potential used up, after seven years operation. This experience and the consequent savings, has been responsible for a long range plan of sludge treatment and ultimate disposal, for the City of Dallas. Generally, the plan consists of building additional "Tertiary Digesters", somewhat removed from present and future population concentrations and in an area which is not subject to flooding.

It should be possible to do a better job this time. We know more about process needs; such as seeding of feed, mixing requirements, scum and odor control, linings of units, that we prefer anaerobic digestion, etc. The plan includes provision for experimenting with liquid dredging of these proposed "Tertiary Digesters", and land disbursement of sludge, from the advance stages of digestion.

DIGESTED SLUDGE LAGOONS

by
DeFro Tossey
Superintendent of Sewage Treatment
Dayton, Ohio

The City of Dayton, Ohio has for over thirty-five years lagooned a major portion of its digested sewage sludge. In the last four years lagooning has been the single method of sludge disposal used. The City of Dayton lagoons are anaerobic in nature. Sludge which is transferred from the regular plant digesters continues to digest over a long period of time and the lower depths of the lagoons contain stabilized material.

The City of Dayton is situated in southwest Ohio on the Miami River. The plant treats an average of approximately 55 million gallons of wastewater per day and the wastewater sludge is from a tributary Dayton population of approximately 300,000 people and complex variety of industrial wastes equivalent to approximately 50% of total sludge load. The sludge digests well without serious difficulties and the digested sludges discharged to the lagoons create no odor or insect problems.

The primary settled sludge is withdrawn to the digesters in a fairly uniform rate of approximately 120 to 125 thousand gallons per day. The solids load in 1969 of 84.700 pounds per day which was 74.7% volatile. The seasonal variations are slight.

The primary and secondary digesters provide a detention period averaging approximately 45 days. The volatile content of the digested sludge transferred to the lagoons averages 50 to 55%.

The standard method of digester operation provides for the return of supernatant from the digesters to the primary plant only if it is of good quality and it is felt it will not be detrimental to our over-all plant efficiency. If good supernatant is not available, then all material from the digesters is transferred to the lagoons. It will be recognized by those familiar with large treatment plant operations that this avenue for disposal of poor quality supernatant provides a means for elimination of one of the most troublesome increments of waste to be handled in many treatment plants.

As stated before, the City of Dayton has used lagoons for disposal of sludge for over 35 years. Many other means of sludge disposal have been used including vacuum filtration with subsequent landfill of sludge, sludge drying beds, and the direct disposal to land by tank truck hauling. This discussion will center around the lagoon operation of the last four years during which time all sludges have been disposed of through the lagoons and irrigation to the land.

The current land disposal facility occupies less than 15 acres of land at the south or rear section of our plant. It is several hundred feet from the nearest public road and is reasonably well isolated from other community developments.

Two lagoons currently being used occupy about six acres and a third lagoon will bring the total to about nine acres. The remaining area is divided into earth cells consisting simply of open ground surrounded by low dykes into which water is pumped from the lagoons in good drying weather. The soil is fine loam and clay with some sand overlaying deep glacial deposits.

The first of these lagoons was placed in service in January 1967 and during the three year period ending in December 1969, had received approximately 96 million gallons of digested sludge. The volume in the lagoon at the end of this time was approximately 14 million gallons which indicates an approximate 7 to 1 reduction in volume through loss of water and further digestion of the solids.

The second lagoon was placed in service in January 1970 in series with the first lagoon. The third lagoon will be complete and on line in early 1971. The three lagoons will then be operated in parallel or in series and are projected to handle all digested sludge through 1973.

Solids accumulation in the lagoon has naturally been steadily increasing. Although no definitive analyses have been made it is estimated that the sludge in the lower levels of the first lagoon has compacted to at least 20 to 25% solids. Digestion continues slowly as evidenced by gas bubbles but no odor problem has resulted. Oil accumulations on the surface come and go periodically and there appears to be a continuing significant digestion of these oils.

It would appear that the success of this lagoon operation without odor and insect problems has been due to the fact that the lagoon has been highly anaerobic at all times. The fact that the sludges prior to entering the lagoons have been through a controlled sludge digestion process in the main plant no doubt contributes to the excellent results obtained in the anaerobic lagoon.

Observations of the lagoon operations in Dayton over many years would indicate that the quality of the digested material being placed in the lagoons is the determining factor as to other problems will develop with this type of operation. Anaerobic lagoons will not necessarily overcome difficulties due to inadequate primary and secondary digesters.

The second part of the system consists of the disposal of the clear water supernatant from the lagoons during good drying weather on open earthen areas adjacent to the lagoons. The water is pumped or siphoned onto these areas and it has been found that literally thousands of gallons per acre per week may be disposed of in this manner. The water which is normally very low in solids but may carry some greases and oils may be channeled onto these plowed areas and the water will infiltrate and evaporate leaving a residual sludge on the surface. In areas where the most intense irrigation of this type has been handled for several years no blinding of the soil has occurred and there has been no sterilizing effect on the soil, on the contrary the materials added to the soil seem to provide good soil nutrient value.

The effect on the underlying groundwater in the vicinity of the lagoons has been of concern and some observations have been made of the water quality in shallow wells. It must be noted that several other factors in the area adjacent to the sewage treatment plant are also potential contributors to the groundwater. These include large landfill operations and dredging operations in the river bottom.

The observation of groundwater quality in the immediate vicinity of the irrigation operation indicates that some disolved matter is carried through the soil into the ground water although extensive definitive analyses have not shown the extent to which these effects are transmitted beyond this immediate area. Extensive groundwater pumping is done in the valley within a mile or so of the plant and there has not been any indicated deterioration of groundwater quality attributed to operations such as that at the sewage treatment plant. These groundwater investigations are scheduled to be expanded in 1970 with a goal of establishing definitive evidence as to the effects on groundwater from this type of operation.

Summary and Conclusion:

The City of Dayton, Ohio has demonstrated through actual performance that large volumes of sludge to be disposed of from heated digesters may be transferred to anaerobic lagoons where extensive further digestion takes place with an eventual stabilization of the remaining solids. It has also demonstrated that the optimum use of land areas connected with this type of operation may be had with the use of infiltration beds adjacent to the lagoons which may be operated during periods of favorable weather and soil conditions. It has also demonstrated that this type operation may be operated throughout the twelve months of the year in a moderately severe climate situation.

A major side benefit of this type of sludge disposal is the fact that the system lends itself to the total disposal of sludges and it is unnecessary to recycle troublesome supernatants, centrates or filtrates in the treatment process.

Dayton has found lagoon disposal to be not only an extremely efficient and economical means of disposal, but satisfactory with regard to effectively returning the residual sludges from wastewater treatment to the soil without destruction of the organics through burning. Dayton has thereby avoided the problems of air pollution incidental to incineration.

EXPERIENCES WITH ANAEROBIC LAGOONS IN AUSTRALIA

by

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It is of historic interest to recall that the first lagoon installation to deliberately take advantage of anaerobic processes to achieve biological purification of wastewater was operated at Werribee in Australia in 1940 thirty years ago and reported in a paper in 1950 (Parker, Jones Taylor "Purification of Sewage in Lagoons" Sew. & Ind. Wastes V.22, 6, 760.)

The definition of an anaerobic lagoon as it was understood at that time was a pond through which raw sewage was passed at a relatively high rate in which sewage solids accumulated and digested and whereby advantage was taken of the process of methane fermentation to stabilize both the deposited sludge and to achieve increased removal of B.O.D. from the supernatant sewage (70-80%).

More recently in Australia increased attention has been given to the minimizing of odour emission from such installations and current practice is directed towards a modification of loading rates to achieve a permanent algal population in the upper layers of such units while still retaining the advantages of anaerobic fermentation in the bottom solids. A condition similar to that recognized by Oswald in his use of the term facultative lagoon. Both types are covered in the present paper.

Current Status of Anaerobic Lagoons in Australia

The treatment of raw sewage in lagoons involving a first anaerobic stage has been principally developed in Australia in the State of Victoria.

In the last ten years the State Rivers and Water Supply Commission the Government Department responsible for approving water and sewage treatment for communities in Victoria outside the Capital City Melbourne, have accepted lagoon treatment as the most suitable form of treatment for the country, cities and towns wherever suitable areas of land are available. As a result several installations treating raw sewage have been constructed in that period. Most of the installations are detailed in Table 1.

Installations Treating Predominantly Domestic Sewage Melbourne

The sewage flow from the population of Melbourne (2,400,000) has been treated at Werribee some 25 miles from the city since 1893. The method used originally was by pasture irrigation but this has been supplemented by grass filtration methods for winter conditions and since 1940 by lagoons for treatment of an increasing proportion of the raw sewage flow.

In the last ten years the area of lagoons has increased from 1100 to 3200 acres making this lagoon complex probably the largest in the world.

The installation consists of a number of separate anaerobic and aerobic pond systems, the shape of individual cells and the number of cells in each system is determined by local topography.

A typical diagram of all lagoon systems is shown in Fig. 1.

In general, design has been based on the investigations previsouly reported (Parker 1959 "Performance of Large Sewage Lagoons" Sew. & Ind. Waste V.31, 2, 133.) The raw sewage flows through numerous parallel lines of ponds with up to eight or ten cells in series in each line. The average area of each cell is 12-15 acres. A header channel feeds by multiple inlets the first cell of each line and the flow then

Table 1. Lagoon installations in victoria treating raw waste

Location	Flow m.g.d. (U.S.)	Composition	Total Lagoon area (acres)	Anaerobic Lagoons		Aerobic Lagoons	
				Area (acres)	No. of cells	Area (acres)	No. of cells
Melbourne							
(a) Werribee	50	Domestic + many industries	3200	500	30	2700	170
Bendigo (pop. 42,000)	3.5	Domestic + abattoirs	47	7.5	3	39.5	5
Ballarat (Wendouree) (pop. 10,000)	0.90	Domestic + potato	17	5	1	12	5
Echuca (pop. 7,500)	0.50	Domestic + milk	43	12	1	31	5
Kerang (pop. 4,100)	0.35	Domestic	2.0	0.5	1	1.5	1
Mooroopna (pop. 3,500)	0.18	Domestic	7.0	1.5	2	5.5	3
Shepparton (pop. 19,000)		Domestic + fruit + veg. cannery + milk + abattoirs					
(a) Wanganui	2.0		100	72	7	28	6
(b) Daldy Road	5.5		250	108	4	142	8
Wangaratta (pop. 16,500)	0.80	Domestic + small abattoirs	43	5.8	2	37.2	8
Wangaratta (Bruck)	1.0	Textile	7.5	7.5	6	-	-
P.R.A.	1.0	Refinery	5.0	1.6	1	3.4	2

gravitates through multiple pipes through the partition banks down through the series of cells. In several instances the final ponds are cross connected to a single long outlet weir. Discharge of all final effluent is to sea into Pt. Phillip Bay.

The flow treated averages 50 m.g.d. (U.S.gals.) but for prolonged periods of high sewage flow this rises to 80-100 m.g.d.

The sewage consists of the domestic flow from about 2,000,000 people plus a wide variety of industrial waste which is responsible for about 15% of the total flow and probably more than half the B.O.D. load.

The lagoon load is distributed to achieve an equal load of B.O.D./ac./day on all the individual systems, the average effluent composition at all times confirms to a Royal Commission 20/30 standard (B.O.D., suspended solids).

The overall loading and performance during winter (water temperature 50°F.) would be as shown in Table 2.

Load and performance during summer conditions (water temperature 70°F.) would be as shown in Table 3.

With such a huge area and large number of individual cells it is a considerable task to maintain an equal distribution of load to all systems. To check this aspect a daily check is made of the sulphide and dissolved oxygen content of each cell in each line of ponds, with the objective of having sulphide reduced to zero at the same stage in each line.

With this multistage series arrangement of ponds, purely anaerobic conditions exist in the first pond with sulphide content of 4-8 ppm. This concentration is comparable with that present in the septic raw sewage reaching the disposal area. The sulphide content drops from pond to pond until dissolved oxygen and algal growth appears. The number of ponds before oxygen appears varies with the load and the time of the year.

In summer the sulphide disappears after only two or three ponds but in winter it may persist through five to six cells.

It is to protect the final effluent from the consequences of temporary overload or unequal distribution that the large series of ponds has been adopted.

Under winter conditions of low water temperature and limited hours of sunlight there appears to be a considerable delay in the changeover from anaerobic to aerobic algal growth, because of this the middle ponds in such a long series do little work. Provision of mechanical aeration of an appropriate stage may well increase the performance of such a type of installation.

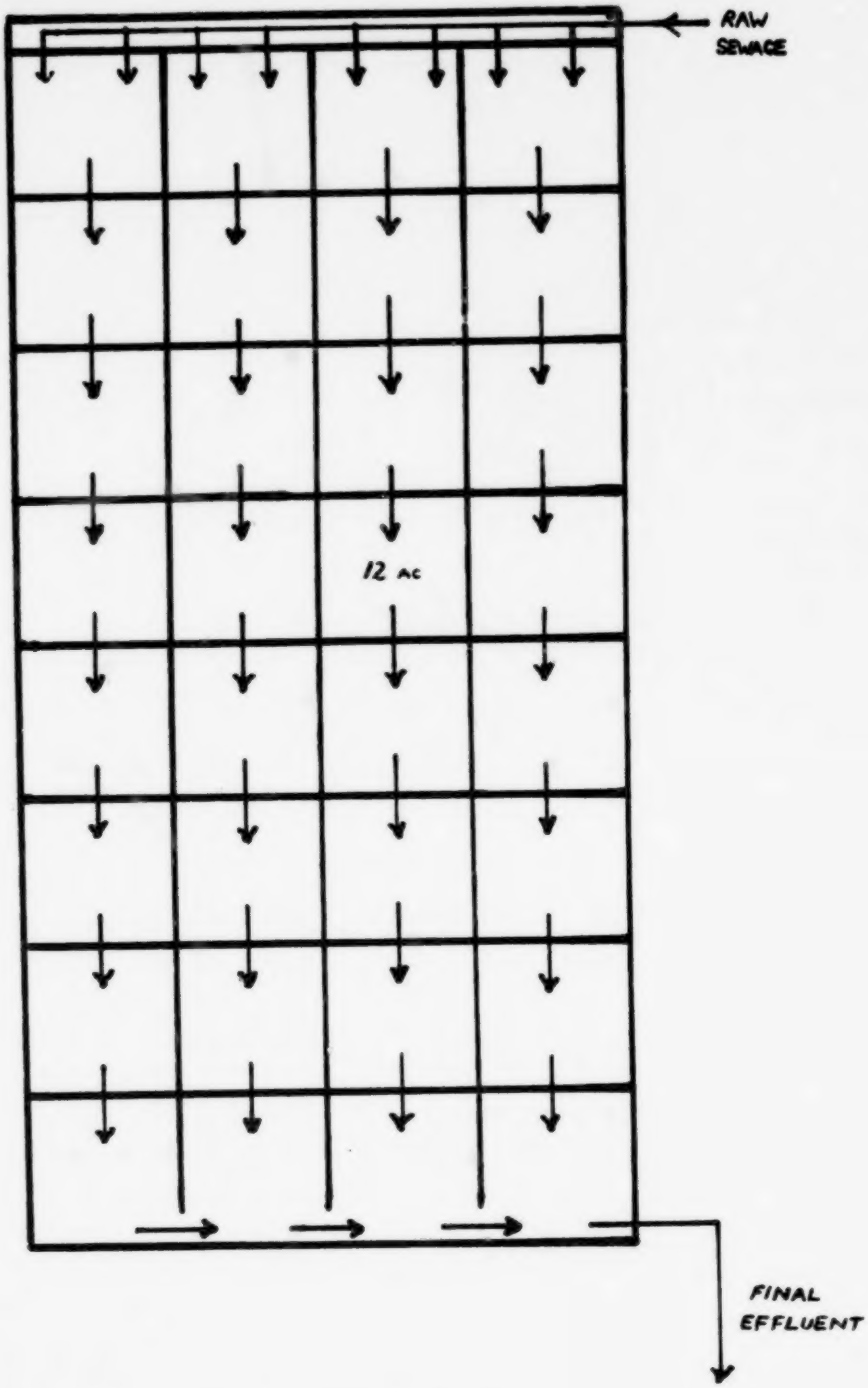
RAW SEWAGE
12 ac
FINAL EFFLUENT

Figure 1. Typical Werribee Layout

Table 2. Lagoon load & performance with Melbourne sewage (winter conditions)

	Flow m.g.d.	B.O.D. ppm.	Raw Sewage lbs/d.	Anaerobic Cells			Aerobic Cells		
				Area acres	Load B.O.D. lbs. ac./d.	Removal %	Area acres	Load lbs. B.O.D. ac/d.	Removal %
Dry weather flow	50	700	280,000	500	560	60	2700	41	93
Wet weather flow	90	500	320,000	500	640	60	2700	41	90

Table 3. Lagoon load & performance (Summer conditions)

Flow m.g.d.	B.O.D. ppm.	Raw Sewage lbs./d.	Anaerobic Cells			Aerobic cells		
			Area Acres	Load lbs. ac./d.	Removal	Area Acres	Load lbs. ac./d.	Removal
50	700	280,000	500	560	75	2700	26	89

Under summer conditions with increased performance of the initial pond and lower B.O.D. of the contents, sulphides do not persist to the same extent and there is a rapid algal development with increased performance of the later ponds.

With heavy loading of solids as well as B.O.D. on the initial ponds, sludge build up occurs in the first two ponds.

Wangaratta (population 16,500)

The sewage flow from this town is mainly domestic apart from a small abattoirs.

The total area of lagoons treating raw sewage is 43 acres with a layout similar to the Melbourne installation, two lines of multistage series units.

The average load and performance are shown in Table 4.

Table 4. Performance of Wangaratta installation

Flow	750,000 g./d
Raw Sewage	B.O.D. 350 ppm
Raw Sewage	B.O.D. 2,100 lbs./day
Lagoon area	43 acres
Anaerobic cells	
Number	2
Area	5.8 acres
B.O.D. load (lbs/ac/d.)	360
B.O.D. removal (%)	75
Suspended solids load (lbs/ac/d.)	360
Aerobic cells	
Number	8
Area	37.2 acres
B.O.D. load (lbs/ac/d.)	15
B.O.D. removal (%)	80

With this small load on the anaerobic cells algal growth is permanently established. The buildup of sludge solids is correspondingly slow, the first line of cells has been in operation 12 years and has not yet been cleaned. The present depth of sludge (10% solids) 30" representing 2,600,000 lbs. of solids or an accumulation of 60,000 lbs/ac./yr. This installation achieves complete purification at all times before discharge of the final effluent to a small creek which is dry in summer.

Mooroopna (population 3500)

The domestic sewage plus the wastewater from a large regional hospital laundry service from this town with a population of 3500 is treated in an anaerobic aerobic lagoon installation with five cells and a total area of seven acres. The first two cells function as anaerobic units and are operated in series - parallel, raw waste being fed directly into both and the effluent from the first discharging to the second and thence through the series.

Load and performance are shown in Table 5.

Kerang (population 4100)

This anaerobic-aerobic installation of two cells is of interest because of the highly mineralized nature of the sewage. The town is in an irrigation area where the drainage is highly saline and the water table is above the level of most of the sewerage system. There is considerable infiltration and the sewage is very weak and high in chloride and sulphate.

The raw sewage has a B.O.D. of 70-100 ppm. and the sulphate sulphur content 50-75 ppm.

Sludge accumulates normally in the first cell but very intense sulphide generation occurs leading to the production of 40 ppm. sulphide. Under these conditions the organic content of the sludge is destroyed in the reduction of sulphate with the production of sulphide and

Table 5. Lagoon Load & Performance Mooroopna

Flow	250,000 g./d.
Raw sewage:	
B.O.D. ppm.	210
B.O.D. lbs./d.	420
Lagoon area	7 acres.
Anaerobic cells	
Number	2
Area	1½ ac.
B.O.D. load (lbs./ac./d.)	280
B.O.D. removal (%)	85
Suspended Solids Load (lbs./ac./d.)	280
Aerobic cells	
Number	3
Area	5½ ac.
B.O.D. load (lbs./ac./d.)	12
B.O.D. removal (%)	90

Table 6. B.O.D. load at Sheparton

	Flow (m.g.d.)	B.O.D. ppm.	B.O.D. lbs/d.	Peak Period of Discharge
Raw sewage	1.5	350	4200	January - Dec.
Abattoirs	0.12	1200	1200	January - Dec.
Butter factory	0.15	6000	7200	Sept. - Dec.
Fruit cannery	2.5	3000	60000	January - April
Soup, Vegetable cannery	1.5	650	7800	February - April
	5.8		80400	

water, rather than by normal production of methane and CO_2.

The high sulphide content of the anaerobic lagoon effluent imposes an oxygen demand additional to the B.O.D. load on the aerobic unit. As a consequence there is induced a massive growth of the red sulphur bacteria, Chromatium and massive blue green algal growth on the surface.

Despite these vicissitudes the final effluent B.O.D. is down to 20 ppm.

This situation points up the difficulties created with highly mineralized sewage and the influence of sulphate concentration on anaerobic lagoon processes.

Installations Treating Domestic Sewage With Predominantly Industrial Waste Component

Shepparton (population 19000)

This town is the centre of a very rich irrigation area, the largest fruit growing area in Australia. Served by the City Sewerage Authority, are a Butter factory manufacturing casein and other milk products with a milk intake of 75000 galls./day an abattoirs slaughtering. 2000 sheep, 400 cattle and 300 hogs per day, the largest fruit cannery in the Southern Hemisphere with a peak daily throughput of 1000 tons of peaches and pears, and a soup, citrus and tomato cannery also with a very large throughput.

The contribution of these various industrial wastes to the B.O.D. load are shown in Table 6.

The butter factory and abattoirs discharge to the domestic sewerage system and this flow is taken by several force mains to the treatment site. The fruit cannery and the soup and vegetable cannery wastes are conveyed by their own separate pipelines to the treatment areas.

The development of effective methods of purification of the two cannery wastes at low cost has been the subject of detailed research and investigation which has already been reported. (Parker C.D. Proc. Purdue Indust. Wastes Conf. 1966 pp. 284-302). These investigations showed that by appropriate mixture of cannery waste and sewage effluent to establish a proper B.O.D./Nitrogen/Phosphate ratio and by regular daily seeding with sewage digested sludge and supernatant liquor these difficult cannery wastes could be effectively treated in anaerobic type lagoons followed either by aerobic type lagoons or oxidation ditches.

With both types of cannery waste it has been shown that algal growth can be retained in the anaerobic type lagoons while sludge digestion continues in the bottom sludge.

Provided these conditions can be maintained these lagoons operate odour free and achieve a B.O.D. removal of 70-85%.

Table 7. Fruit cannery waste treatment Shepparton - summer 1968.

	Anaerobic lagoon S_2 (9 ac.)	Anaerobic lagoon S_3 (10 ac.)
Flow m.g.d.	0.45	0.50
B.O.D. (ppm.):		
influent	1960	1960
effluent	374	346
B.O.D. load lbs./ac./d.	878	790
Removal %	81	82
Algae	Present	Present
Odour level	Very slight	Very slight

Table 8. Vegetable cannery waste treatment at Shepparton (General products & tomato)

	Summer 1965		Summer 1967	
	Anaerobic Lagoon No.1 (5ac.)	Anaerobic Lagoon No.2 (5ac.)	Anaerobic Lagoon No.1 (5ac.)	Anaerobic Lagoon No.2 (5ac.)
Flow m.g.d.	0.53	0.53	0.58	0.58
B.O.D. (ppm.):				
influent	728	728	296	296
effluent	187	139	57.0	51.1
B.O.D. load lbs./ac./d.	628	628	276	276
Removal (%)	74	80	80.5	82.6
Algae	Absent	Absent	Present	Present
Odour level	Present	Present	Nil	Nil

Table 9. Vegetable cannery waste treatment Shepparton

	Winter 1964		Winter 1965	
	Anaerobic Lagoon No.1 (5ac.)	Anaerobic Lagoon No.2 (5ac.)	Anaerobic Lagoon No.1 (5ac.)	Anaerobic Lagoon No.2 (5 ac.)
Flow m.g.d.	0.4	0.41	0.51	0.51
B.O.D. (ppm.):				
influent	939	939	400	400
effluent	234	248	59	49
B.O.D. load lbs./ac./d.	625	625	135	135
Removal (%)	75	75	85	87
Algae	Absent	Absent	Present	Present
Odour level	Present	Present	Absent	Absent

The maximum loading to retain these conditions is different for the two different cannery wastes. The detailed conditions are shown in Tables 7, 8 and 9.

It will be seen that for fruit cannery wastes the maximum loading is 600-800 lbs. B.O.D./ac./d. (with lagoon depth of 4½ ft.) and the lagoon contents have a B.O.D. of 400-600 ppm. In the case of the vegetable cannery the maximum permissible loading in summer is 350 lbs. B.O.D./ac./d. to achieve a B.O.D. of the lagoon contents of 80-100 ppm. and in winter a load of 150-200 lbs./ac./day and a B.O.D. of the lagoon contents of 80-100 ppm. Provided these conditions are observed the ponds are always green and odour free.

Supervision of operation consists of twice weekly checks of pH and dissolved oxygen, with weekly laboratory examination for B.O.D., suspended solids, pH, algal population. Distribution of load between the various cells is determined from these observations.

The installation consists of three separate sections.

The domestic sewage with abattoirs and milk waste is

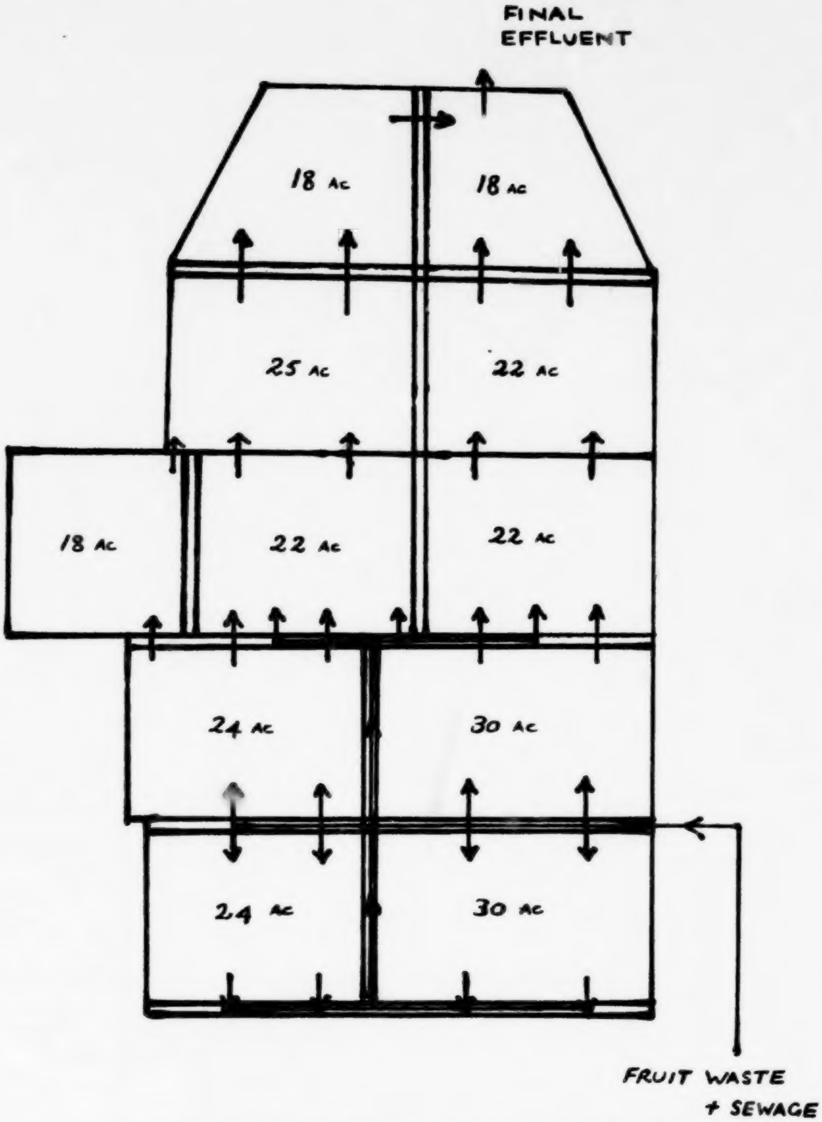

Figure 2. Daldy Road Installation, Shepparton (250 Acres)

settled in conventional primary clarifiers and the sludge digested in heated digesters. The supernatant and digested sludge from the digesters together with settled sewage is mixed with fruit cannery waste and travels by gravity through a 30" concrete sewer to the Daldy Road installation where it is treated in 108 acres of anaerobic (6-8 ft. deep) followed by 142 acres of aerobic (3-4 ft. deep) lagoons in three series of pond. The layout is as shown in Fig. 2.

During the period May-December there is no cannery waste and the ponds operate at a low B.O.D. loading treating settled sewage, milk and abattoirs wates.

Until this season fruit cannery flow has been partially treated in temporary facilities at the Wanganui Road site. Only part of the flow has been fully treated, the remainder only receiving anaerobic stage treatment to a B.O.D. of 400-600 ppm.

The vegetable and soup cannery waste is conveyed to a separate site at Wanganui Road where it is mixed with settled sewage, digested sludge and supernatant and treated in 35 acres of anaerobic followed by 28 acres of aerobic lagoons. At this site there has also been available a further 37 acres of anaerobic lagoon to temporarily treat fruit cannery waste. All ponds at the Wanganui Road site are 4 ft. deep. the layout is as shown in Fig. 3.

Detailed examinations have been made for sludge accumulations in all anaerobic ponds. This has shown that in the ponds which have treated the soup waste for nine years the sludge depth averages 6". In the temporary ponds which have treated fruit cannery waste for five years the average sludge depth is 5". So far there has been no need to remove sludge from any ponds.

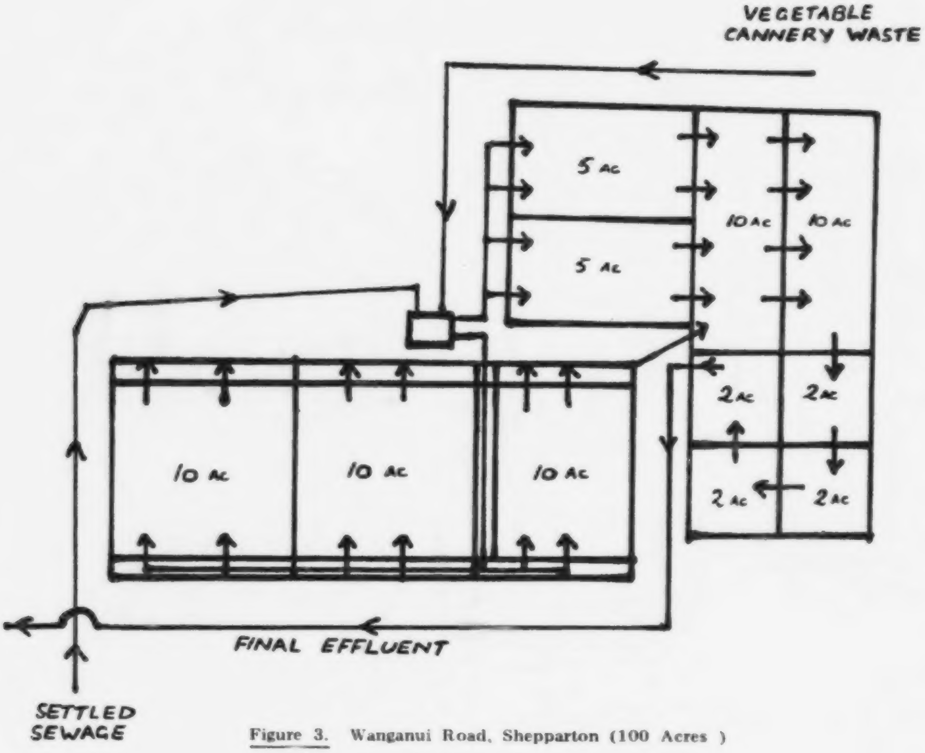
VEGETABLE CANNERY WASTE
5 AC
5 AC
10 AC
10 AC
2 AC
2 AC
2 AC
2 AC
10 AC
10 AC
10 AC
FINAL EFFLUENT
SETTLED SEWAGE

Figure 3. Wanganui Road, Shepparton (100 Acres)

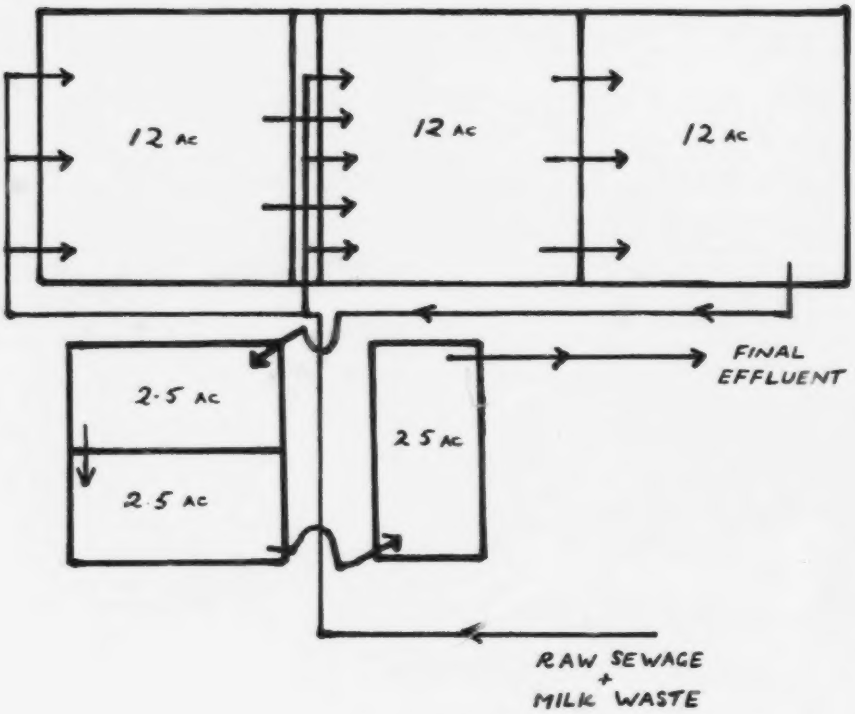
12 AC
12 AC
12 AC
FINAL EFFLUENT
2·5 AC
2·5 AC
2 5 AC
RAW SEWAGE + MILK WASTE

Figure 4. Echuca

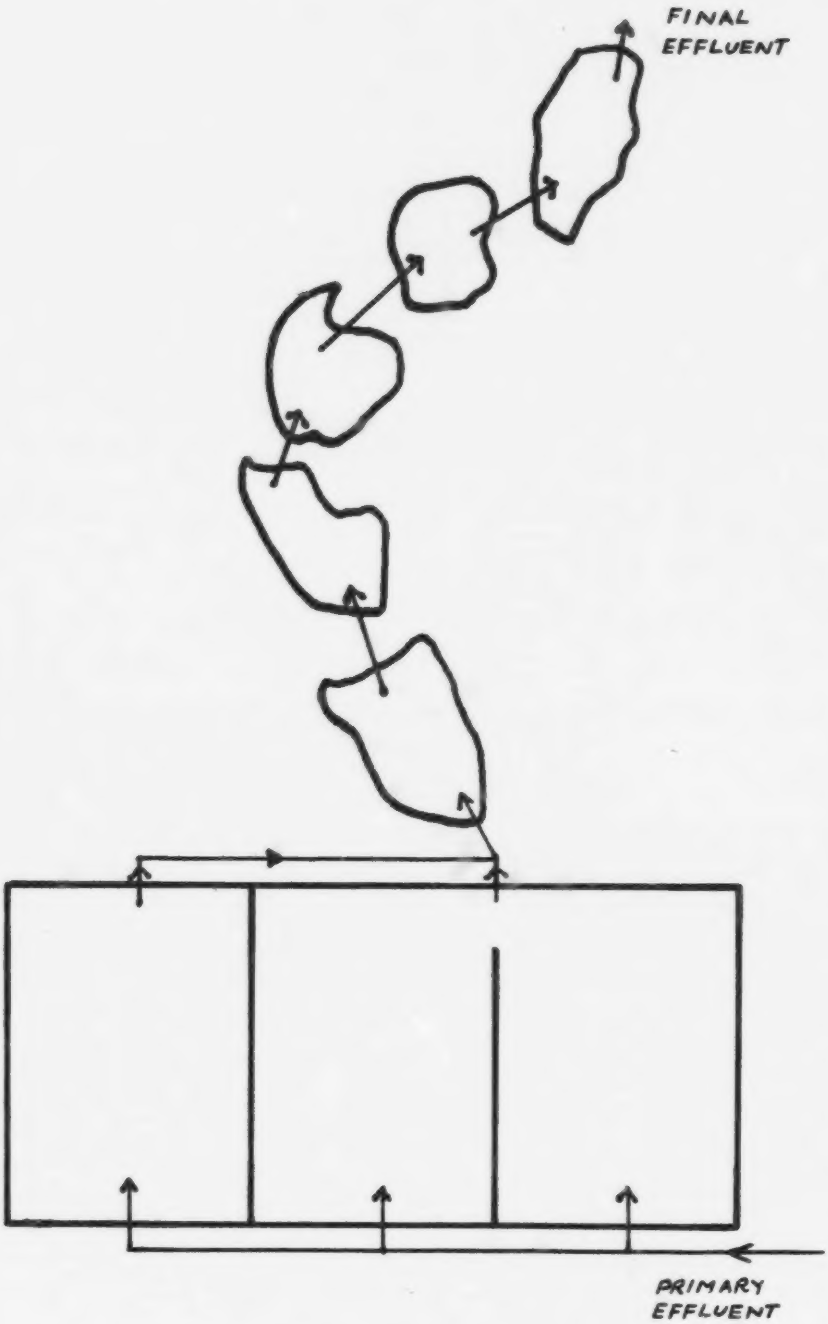
FINAL EFFLUENT
PRIMARY EFFLUENT

Figure 5. Bendigo

Table 10. Performance of Echuca lagoons

	B.O.D. ppm.	Suspended Solids ppm.	pH
Raw sewage	252	288	7.1
Lagoon 1 effluent	35	83	7.2
Lagoon 2 effluent	30	70	7.5
Lagoon 3 effluent	12	55	8.2
Lagoon 5 effluent	35	80	7.4
Lagoon 6 effluent	13	45	7.8

Table 11. Performance of Bendigo Lagoons

	Flow m.g.d.	B. O. D.		
		ppm.	lbs./d.	lbs./ac./d.
Raw sewage (combined)	2.25	430	7800	
Slaughter house waste	0.20	1180	1800	
Settled sewage	2.25	196	3600	170
Anaerobic lagoon effl.		95		
Final aerobic effluent		23		

Echuca(population 7500)

Here the city domestic sewerage system has a large butter factory discharging to it. The B.O.D. load from both domestic and milk wastes are each about 1200 lbs./day the daily flow is 500,000 gals./day.

The combined raw waste flow is treated through a 6-cell anaerobic-aerobic system of 43 acres. The layout is as shown in Fig. 4.

The first two cells may be operated series-parallel, the raw waste being adadded to either or both and the effluent from the first cell flowing to the second and then in series through the remaining four cells.

This installation has only been in operation eight months. Prior to this the town was served by an overloaded trickling filter plant with primary clarification and separate unheated digesters. This plant had become grossly overloaded by the addition of the milk waste and was susperseded by the lagoon installation.

During the establishment of efficient operation of the anaerobic lagoon units the primary clarification and sludge digesters has continued to treat about one quarter the flow to provide a continuous supply of supernatant liquor and digested sludge to seed the anaerobic lagoon units.

The average of the results of analysis through the lagoon system over the last month are as shown in Table 10.

The B.O.D. load on the first two anaerobic lagoons which have been operated in parallel has been 100 lbs./ac./d. and the overall load on the whole installation 60 lbs./ac./d.

This installation has been deliberately designed to receive a light load initially as this is the first such installation constructed in Victoria to take large quantities of milk wastes, based on investigations described later.

It also had the advantage of providing, if experimental findings were confirmed, reserve capacity for the expected expansion of the milk waste load.

Bendigo (population 42,000)

The sewage from this city contains a flow of 200,000 gals. per day of slaughter house waste.

The combined flow of 2.25 m.g.d. is given partial primary sedimentation and then treated through three anaerobic lagoons all operated in parallel and 6 cells of aerobic lagoon of irregular shape. The total area is 47 acres and the layout as shown in Fig. 5.

The details of composition, flow and loading are as shown in Table 11.

Ballarat (population 60,000)

Wendouree Plant (population 10,000

This plant originally designed for a domestic population of 7500 now has connected 10,000 persons and the discharge from a large instant mash potato factory

The respective sewage and industrial waste flows are: -

	Flow (g./d.)	B . O . D.	
		ppm.	lbs./d.
Domestic	500,000	300	1200
Potato waste	125,000	605	605
	625,000		1805

The original plant consisted of primary sedimentation with a single unheated digester, two trickling filters and four tertiary lagoons in series, with a total area of 7 acres.

On connection of the potato plant it was found that the acidified raw sludge could not be handled in the unheated digester although oxidation through the two filters and four tertiary lagoons brought the B.O.D. down to 10-15 ppm.

To cope with the overload on the digester two additional lagoons each of 5 acres were constructed the first of which functioned as an anaerobic lagoon to take raw sludge. The layout and operation of the plant is shown diagrammatically in Fig. 6.

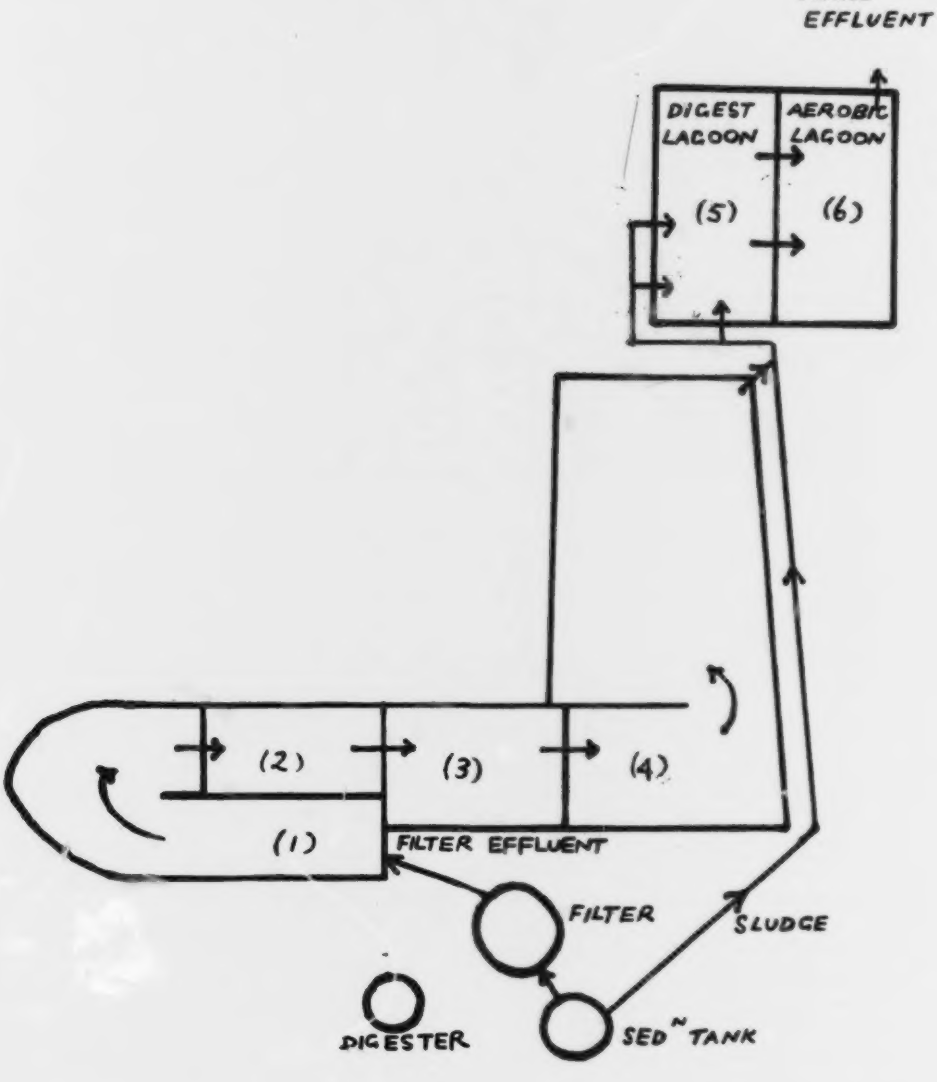
FINAL EFFLUENT
DIGEST LAGOON
AEROBIC LAGOON
(5)
(6)
(2)
(3)
(4)
(1)
FILTER EFFLUENT
FILTER
SLUDGE
DIGESTER
SEDⁿ TANK

Figure 6. Ballarat (Wendouree)

Table 12. Performance of Ballarat lagoons

	B.O.D. ppm.	Suspended Solids ppm.	pH	Nitrogen Org.-N. ppm.	NH$_3$N. ppm.	NO$_3$	Tot.N. ppm.	Phosphate PO$_4$ ppm.
Raw sewage	256	204	5.8	14.5	40	0	54.5	35.0
Settled sewage	170	80	5.6	10.0	40	0	50.0	32.0
Filter effluent	13.5	68	6.8	5.1	15	21	41.1	28.0
Lagoon 1.	10.0	30	7.1	4.0	14.5	8.0	26.5	20.0
Lagoon 4.	15.0	16	7.3	2.3	14.5	7.3	24.1	15.0
Lagoon 5 (anaerobic)	16.0	32	7.0	4.3	10.5	2.6	17.4	16.0
Lagoon 6 (aerobic)	17.0	24	7.4	3.0	15.0	1.9	19.9	17.0

Table 13. Wangaratta Textile waste lagoons

	B.O.D. ppm.	Suspended Solids ppm.	pH	Sulphide ppm.
Raw waste	147	109	6.1	0.7
Anaerobic lagoon effluent	87	72	7.2	2.5
Grass effluent	28	58	7.2	0

Table 14. Solids accumulation

	Flow/ac/ per year mg.	Suspended Solids		Solids Deposited lbs/ac/yr.	Sludge Accumulation		
		Inf.	Eff.		Depth per year	lbs/ac/yr.	Reduction %
Werribee	75	550	150	240,000	6"	136,000	40
Wangaratta	90	350	150	145,000	2½"	60,000	6[illegible]
Shepparton	30	250	50	50,000	0.7"	12,000	75

The combined sewage and potato waste flows to the primary clarifer and the whole of the clarified flow is distributed to the two filters and then to the original four lagoons.

The effluent from the final lagoon of these four cells then flows to the first of the two new cells recombined with raw sludge drawn from the clarifier for four days of the week. The remaining three days (Saturday, Sunday and Monday) the sludge is drawn normally to the digester.

The flow from this digestion lagoon passes to a final aerobic lagoon and then to the Creek.

By the use of a digestion or anaerobic lagoon at this stage the load is taken off the digester and the load in the lagoon is such that the solids are adequately digested in the lagoon while maintaining an algal content and level of dissolved oxygen throughout the year. The final effluent is always of acceptable quality. Typical results are shown in Table 12.

Installations Treating Industrial Waste
Wangaratta(textile waste)

In this city there are two textile factories one spinning and dyeing synthetic fibres and the other weaving and dyeing a wide range of fabrics.

The combined waste from the two factories is treated through & a 6-cell shallow anaerobic type lagoon of 7.5 acres 16" depth and the effluent allowed to flow over grass. The flow is 1,000,000 gals./d. and the detention time 2 days.

The B.O.D. of the waste is brought down to acceptable levels but the colour remains high.

Conditions in the lagoon are entirely anaerobic despite the low B.O.D. of the incoming waste. Sulphide is generated and no algal growth occurs.

Typical analyses are as shown in Table 13.

The B.O.D. load on the lagoon system is 142 lbs./ac./d. and the removal 56 lbs./ac./d. There has been significant odour developed from this system and it is to be replaced by an oxidation ditch.

Petroleum Refineries (Aust.) Ltd.

The refinery effluent with a daily flow of 1,000,000 g./d. is treated after passage through a A.P.I. separator in a lagoon installation of 4 cells with a total area of seven acres.

It is found that by the addition of trace metals magnesium, calcium, manganese, and cobalt, a profuse algal growth can be maintained in all cells and the B.O.D. is reduced from 80-100 ppm. to 5-10 ppm. and phenols are completely destroyed with seven days detention.

Research in Austrailia
1. Basic Studies

In these laboratories under U.S.P.H.S. research grant WP-00321, comprehensive study has been made of the function of solids in lagoon treatment of wastewater. This work has been reported in the paper (Parker & Skerry, "Function of Solids in Anaerobic Lagoon Treat. of Wastewater" J.WPCF Vol. 40, No. 2, 193, 1968). These studies have confirmed earlier observations that the presence of accumulated sludge solids in anaerobic lagoons is essential for maximum B.O.D. removal.

A standardized procedure has been developed to

Table 15. Loadings for facultative conditions

Waste	Summer (70°F.) lbs./B.O.D./ac./d.	Winter (50°F.) lbs/B.O.D./ac./d.
Sewage	400	150
Fruit cannery waste	800	—
Vegetable cannery waste	350	150

Table 16. Effect of B.).D./nitrogen ratio on performance of anaerobic lagoons treating fruit cannery waste

Lagoon	Flow	B.O.D. ppm.		B.O.D. load lbs/ac./d.	Removal		B.O.D./N. Ratio
		Inf.	Eff.		lbs/ac./d.	%	
T_2	0.43	1480	528	339	218	64	116
S_2	0.50	1960	374	878	710	81	45
S_3	0.50	1960	346	790	651	82	45

determine the relative fermentation ability of different sludges using a synthetic sewage and standard conditions of flow and sludge content.

The biochemical properties of sludges vary in different parts of a pond and in different installations. The powers of methane fermentation appear to not necessarily run parallel with the ability to reduce B.O.D.

The rate of solids accumulation has been determined at a number of field installations and an attempt made to relate solids loading to rate of accumulation. Typical values are as shown in Table. 14.

The interrelationship of sludge solids accumulation and algal development in moderately loaded anaerobic units has been examined in relation to B.O.D. loading and it has been found that under the same temperature conditions (summer 70°F., winter 50°F.) the permissible loading for coexistence of sludge digestion and algae, differs with different types of waste. The predominant algal are also different.

The permissible loadings are as shown in Table 15.

Operation of anaerobic units in which the nutrients have been supplied from raw sewage without digester seed and with varying ratio of sewage to cannery waste have demonstrated as shown in Table 16, that with anaerobic lagoons the C/N/P ratio has a considerable influence on performance. Currently investigations are being evaluated to determine the extent to which the nutrient can be supplied as inorganic salts. It has been shown that even with ponds already well supplied with digested sludge, the daily addition of digested sludge and digester supernatant does double the performance.

3. Anaerobic Lagoons to Treat Milk Wastes

Sponsored by the Australian Dairy Produce Board, investigations have been carried out by these laboratories to establish low cost methods of treating wastes from milk product manufacture in Australia.

The wastes consist of those from butter making, spray and roller dried powdered milk, cheese and casein manufacture. The wastes fall into two categories, the general factory wastes consisting of floor and wash water from some or all these operations and the concentrated wastes principally whole whey from cheese and casein manufacture.

Using experimental ponds of ½ acre in area it has been shown that unneutralized factory waste can be loaded into anaerobic type lagoons to maintain an algal population, at a spring B.O.D. loading of 200 lbs./ac./d. under peak load conditions provided regular sludge additions are made. These ponds are odour free and achieve 70-80% removal of B.O.D. Complete purification can be achieved by further treatment of the effluent in aerobic units or in oxidation ditch.

It has also been shown that unneutralized whole whey can be treated by regulated addition to an anaerobic type lagoon with an algal population, provided the B.O.D. loading is restricted to 400 lbs./ac./d.

Summary of Australian Practice Concerning Anaerobic Lagoons

Layout

Many of the installations particularly the large Melbourne system adhere to earlier practice of an anaerobic/aerobic area ratio of about 1/5.

Recent installations particularly those treating industrial wastes with sewage and those where particular attention is given to odour free operation have tended to adopt a ratio near 1/1 with a view to minimizing the load on the anaerobic cells. In most cases the aerobic cells consist of several small units in series.

Depth

Most installations have been built with a uniform depth of about 3-4 feet for both anaerobic and aerobic units.

Recently some anaerobic units have been built 6-8 feet deep to reduce area and provide increased sludge storage capacity although it must be recognized that if area is reduced proportionate to increased depth, the rate of sludge depth accumulation will be the same as for shallower ponds. A recent large installation has shown that where land value is $300 per acre, the increased cost of constructing deeper cells offsets the saving in land cost.

Loading and Performance

Loading for sewage has been held at 400 lbs./ac./d. for winter conditions for purely anaerobic conditions although much higher loadings are permissible in summer. For continuous facultative conditions through winter a load of 250 lbs./ac./d. is used.

For combined treatment with food industry wastes permissible loadings vary considerably, for summer conditions these vary from 400-800 lbs./ac./day to maintain facultative conditions.

Most purely anaerobic ponds are operated to achieve a B.O.D. reduction of 60-75%. When loaded to achieve facultative conditions a reduction of 70-85% is achieved.

B.O.D./Nutrient Ratio

It is found that for cannery wastes a B.O.C./nitrogen ratio of 50/1 will achieve the performance described. Increase of this ratio to 150/1 reduces the performance threefold.

Control and Supervision of Operation

In the larger installations, regular frequent on site checks are made for pH, dissolved oxygen and sulphide in addition to laboratory examination for B.O.D., suspended solids and other sanitary analyses.

Sludge Accumulation and Removal

Sludge appears to accumulate to the extent of 25-60% of the solids deposited and this can be calculated from inflow and outflow suspended solids. The ratio of solids built up depends on the nature of the waste, the rate of accumulation is much less with highly carbonaceous wastes such as those from canneries.

Removal of sludge solids where it has been practised has been best effected by pumping the wet sludge to drying areas. In large installations this has been expedited by having a deep transverse sump across the lagoon into which the sludge will flow or can be pushed and from which it can be pumped.

Odour

Lagoons operated at high rates do show odour, if it is sewage this shows as sulphides in the pond contents although the pH does not drop much below 6.5.

With fruit and mild wastes excessive load produces a considerable drop in pH (to 5.5 or less), the presence of high volatile acids but no sulphides are found under these conditions.

Provided the loadings mentioned above are adhered to anaerobic type lagoons can be operated with a very low level of odour.

ANAEROBIC LAGOON TREATMENT OF PACKINGHOUSE WASTEWATER

Mark J. Hammer and C.Dale Jacobson

Introduction

Meat-Processing Wastewaters[1,2]

A slaughterhouse is principally a killing and dressing plant which does little or no processing of by-products. Packinghouse operations, in addition to slaughtering, normally involve cooking, curing, smoking and pickling of meat, manufacture of sausage, and rendering of edible and inedible fats. Wastewater sources include: killing floor, carcass dressing, casing cleaning, rendering, hog hair removal, hide processing, meat processing, cooling room, general cleanup and animal holding pens.

The quantity of waste resulting from slaughtering and meat packing depends to a considerable extent upon the methods used for the recovery and processing of by-products. Blood recovery for a salable by-product is commonly practiced. Normally, paunch wastewater is separated and screened to remove coarse solids. In older plants paunch material may be washed down the sewer. In the most recent waste-saving process, paunch contents are handled dry and hauled away for disposal without washing. Rendering can be performed by wet or dry processes. Current practice for edible rendering is either dry or wet with evaporation of tank water, while inedible rendering is generally dry. Wet rendering without evaporating the tank water creates the greatest pollutional load.

In addition to blood recovery, paunch manure removal and low-pollutional rendering, many meat-processing plants skim their raw wastewater in a holding tank. The settled solids and skimmed grease are processed by inedible rendering. A properly designed settling-skimming tank can reduce wasteloads by approximately 25 percent. A minimum detention time for economic grease recovery is 20 minutes,[2] however, for maximum waste strength reduction the detention time should be at least 40 minutes. Air flotation, although not commonly used at meat-processing plants, is effective in removing grease and suspended solids.

Table I is a listing of wasteloads after grease skimming with reference to process technology based on questionnarie data from approximately one-half of the federally inspected slaughterhouses in the U.S.[2] Typical wastewater characteristics from meat-processing plants are listed in Table II.

Meat-processing plants usually operate 5 days per week for a period of 12 to 18 hr per day. The bulk of

Table 1. Average BOD load and wastewater flow from meat-processing plants after settling-skimming tanks (Ref. 2, pp. S-4 & 32)

Type of technology	Percentage of plants 1966	lb BOD / 1000 lb LWK*	Gallons / 1000 lb LWK	BOD mg/1
Old	10	20.2	2112	1150
Typical	80	14.4	1294	1350
Advanced	10	11.3	1116	1210

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*LWK = Live Weight Kill

Table 2. Typical slaughterhouse wastewater characteristics

Parameter	Common Range	Typical Values
Flow, gal/1000 lb LWK	500-2000	1200
BOD, mg/1	1000-2000	1400
Suspended solids, mg/1	400-2000	1000
Grease, mg/1	200-1000	500
Total nitrogen, mg/1		150
Temperature, °F	80-90	82
pH	6.8-7.5	7.0

processing and cleanup waste flows occur during operating periods on weekdays. Peak hourly flows may vary between 200 and 300 percent of the average daily flow. Weekend wastewater flows are generally limited to cleanup and maintenance operations and cooling water.

Anaerobic Treatment

Anaerobic digestion is carried out by a wide variety of bacteria categorized into two groups, acid-forming bacteria and methane-forming bacteria. The acid-formers are facultative or anaerobic bacteria that metabolize the organic matter forming organic acids as an end product, along with carbon dioxide and methane. The acid-splitting methane-formers use organic acids as substrate and produce the gaseous end products of carbon dioxide and methane. These methane bacteria are strict anaerobes inactivated by the presence of dissolved oxygen and inhibited by the presence of oxidized compounds. They are sensitive to variations in pH and other environmental changes.

The two major groups of bacteria must operate cooperatively to perform gasification of organic matter. The first stage produces food (organic acids) for the second stage, while the second stage consumes these organic acids preventing excess acid accumulation. In addition to producing food for the methane bacteria, the acid-formers also reduce the environment to one of strict anaerobiosis by using oxidizing compounds and producing reducing agents, such as hydrogen sulfide.

Advantages of anaerobic treatment in comparison to aerobic treatment are: high degree of waste stabiliztion is possible; low production of waste biological sludge; low nutrient requirements; no oxygen requirements; and methane produced is a useful end product.[3] Disadvantages of anaerobic treatment are: incomplete metabolism of waste organics; and relatively high temperature requirements.

Optimum environmental conditions for anaerobic digestion include: an operating temperature in the general range of 85° - 95°F; pH in the range of 7.0 - 7.1; strict anaerobiosis; complete mixing; and a solids retention time of 10 to 20 days.

Important characteristics for a wastewater to be amenable to anaerobic treatment are: high organic strength, particularly in proteins and fats; sufficient inorganic biological nutrients; adequate alkalinity; relatively high temperature; and freedom from toxic materials. Meat-processing wastewaters have these characteristics.

Anaerobic Treatment Processes

A schematic diagram of the anaerobic contact process is shown in Fig. 1. Full-scale anaerobic contact processes have been in operation since 1959.[1,4] The influent wastewater flow is equalized over a 24-hr period by storage in a tank. After preheating to 95°F, the wastewater is fed at a uniform rate to the complete mixing digesters. Detention time in the digesters is approximately 12 hr with a mixed liquor suspended solids concentration of 7,000 to 12,000 mg/1. Mixed liquor flows through vacuum degasifiers to gravity sludge separators. Detention time in the separators is about 1 hr based on total flow. Settled sludge is removed by suction-type removal mechanisms and recycled at a rate of approximately three volumes per volume of incoming

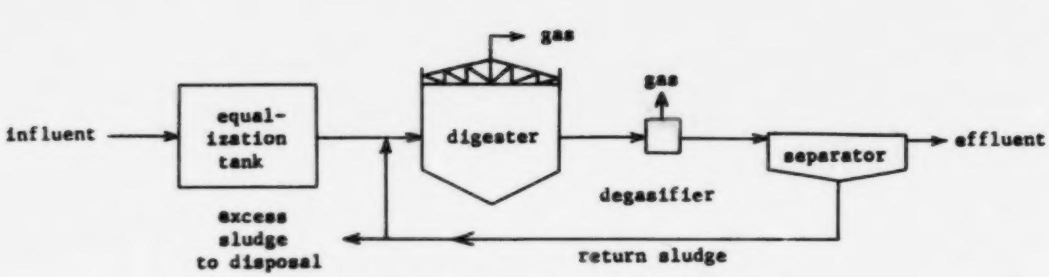


Fig. 1. Schematic diagram of the anaerobic contact process.

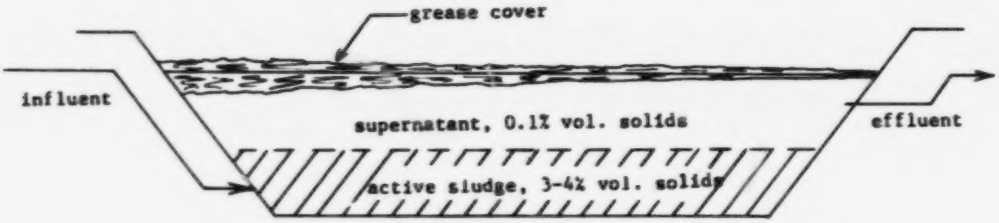

grease cover
influent
supernatant, 0.1% vol. solids
effluent
active sludge, 3-4% vol. solids

Fig. 2. Schematic diagram of an anaerobic lagoon.

raw wastewater. Excess sludge is wasted to a disposal lagoon, and the wastewater effluent flows to oxidation ponds for final polishing. BOD removal in the anaerobic contact process is in the range of 96 to 98 percent.

A schematic diagram of an anaerobic lagoon is illustrated in Fig. 2. Several full-scale anaerobic lagoon systems for treating meat-processing wastewaters have been constructed and evaluated in the last 10 years.[4,5,6,7,8,9]. Influent wastewater flows directly into the anaerobic lagoon without prior equalization. Raw wastewater enters near the bottom and mixes with the suspended active microbial solids in the sludge blanket. The discharge pipe is located on the opposite end of the lagoon submerged below the liquid surface. Excess grease floats on the liquid surface forming a natural cover for retaining heat and maintaining anaerobic conditions. At a BOD loading of 15 lb BOD/1000 cu ft/day the liquid detention time is approximately 6 days. Equalization, digestion and sedimentation all take place in the anaerobic lagoon, whereas, in the anaerobic contact process each of these operations are performed in a separate tank. Excess solids are wasted in effluent from a lagoon, therefore, process efficiency is lower than in the anaerobic contact process where excess sludge is wasted to disposal. BOD efficiency of anaerobic lagoons is generally in the range of 70 to 90 percent.

First stage lagoons using over-and-under baffles have been constructed for treating poultry processing wastewater.[10] The baffles apparently enhance lagoon treatment by providing a series of settling compartments, and top to bottom mixing. Baffling has not been used in anaerobic lagoons treating slaughterhouse wastes since the baffles would interfere with formation of a grease cover.

The latest anaerobic treatment process being evaluated is anaerobic filtration. Results of laboratory tests using upflow anaerobic filters indicate that the process may have several advantages in treating soluble wastes: high solids retention times can be achieved without recirculation; satisfactory operation is possible at lower temperature; and intermittent operation is possible with rapid restarting when wastewater flow is restored.[11]

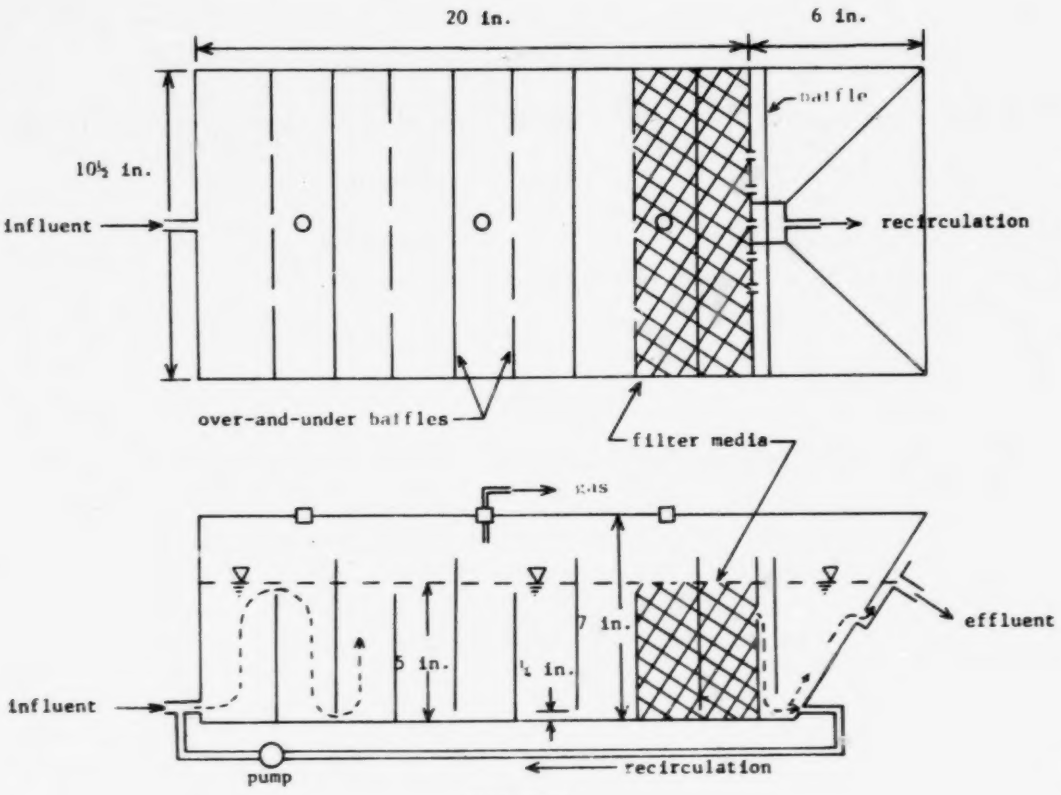
Laboratory Anaerobic Lagoon Study

The following laboratory study, performed at the University of Nebraska, was conducted in an attempt to evaluate anaerobic lagoon treatment of packinghouse wastewater.

The laboratory anaerobic digester, shown in Fig. 3, consisted of an enclosed plastic tank with a liquid volume of 15.9 liters. The unit was provided with: plastic over-and-under baffles, a recirculation return line from the final compartment to the influent, a gas-tight cover with a line to a separate gas collector, and accessory holes through the cover to measure pH and temperature. The tank was placed in a hot water bath to maintain a constant digestion temperature of 79°F.

The feed was settled and skimmed packinghouse wastewater held at refrigeration temperature. A positive displacement pump was used to pump continuously-mixed feed wastewater from the refrigerated jug to the tank at a constant rate. Effluent from the tank returned by gravity to a refrigerated storage container.

The seed sludge used to start the anaerobic unit was a screened, diluted, anaerobic, digesting sludge from a


20 in.
6 in.
10½ in.
influent
baffle
recirculation
over-and-under baffles
filter media
gas
7 in.
5 in.
¼ in.
influent
effluent
pump
recirculation

SIDE VIEW

Fig. 3. Laboratory Anaerobic Tank.

Table 3. Data from laboratory anaerobic lagoon study settled and skimmed packinghouse wastewater

BOD Loading lb BOD/day 1000 cu ft	Hydraulic Detention Time days	Gas Production cu ft/day lb BOD applied	Removal Efficiencies BOD %	COD %
Anaerobic Tank without Filter Media				
18	5	6.3	84	88
65	3	0.7	Impending	Failure
Anaerobic Tank with Filter Media				
20	4.8	3.3	89	95
17	3.8	3.0	75	91
40	2.4	3.6	87	95
58	0.9	3.3	75	80

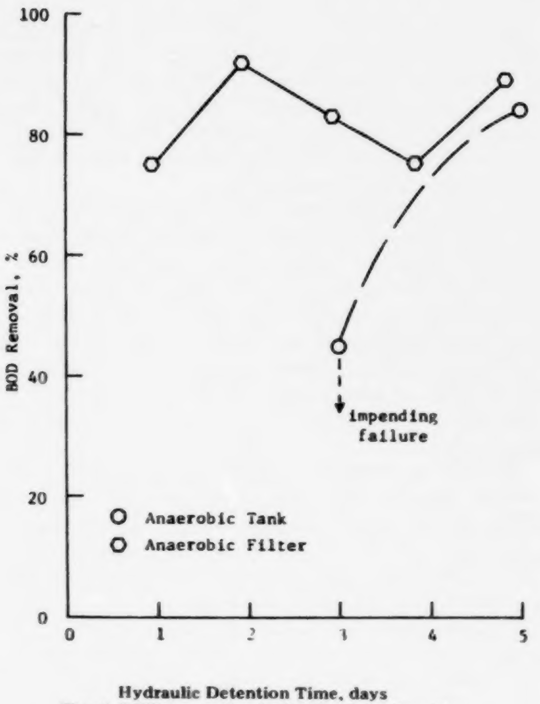


Fig. 4. BOD removal vs hydraulic detention time.

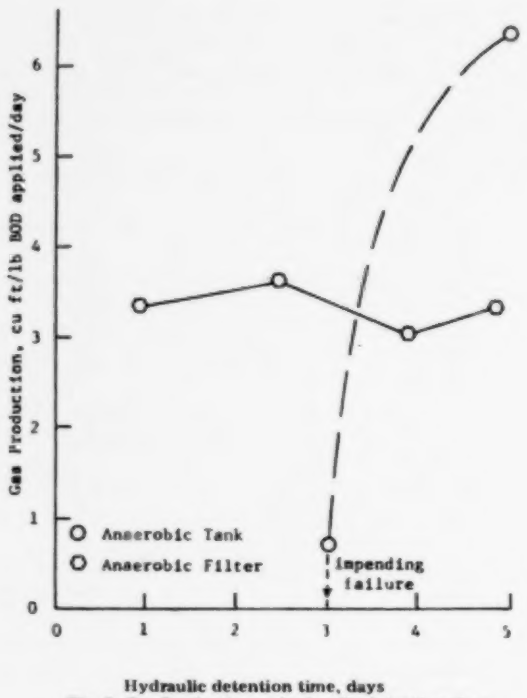


Fig. 5. Gas Production vs hydraulic detention time.

high-rate sewage sludge digester. Initial operation of the tank was tested using a soluble high-strength organic wastewater. The unit did not function without the over-and-under baffles due to short-circuiting. With the baffles in the tank, a high degree of anaerobic digestion of the soluble wastewater was achieved at a detention time of 3 days. Recirculation was used to improve mixing of the tank contents.

After these preliminary tests using the soluble organic wastewater, the feed was changed to a settled and skimmed wastewater from a local packinghouse plant. For the initial run, the wastewater was applied at a loading of 18 lb BOD/1000 cu ft/day with a 5 day detention time. It was observed during this initial run that recirculation to improve mixing was actually detrimental to the process rather than enhancing it. Raw suspended solids were being carried through the tank too rapidly and appeared in the effluent. Under steady state conditions at a detention time of 5 days without recirculation the BOD removal efficiency was 84 percent (Table III).

The feed rate was then increased reducing the detention time to 3 days. Gas production began to decrease after the first day at this higher feed rate. After three days, the effluent became odorous, contained raw solids, and the pH of the tank contents was dropping. The anaerobic process was rapidly approaching failure.

Failure in the baffled tank appeared to result from the loss of active solids, too short a solids retention time. At this time, the last two baffled compartments (Fig. 3) were filled with ½ in. carbon Raschig rings having a free gas space of 74 percent and 114 sq ft of surface area per cu ft.

The remainder of the experimental runs were performed using the baffled tank with two compartments filled with the filter media. The results are shown in Table III. Experimental runs with filter media were conducted at hydraulic detention times of 4.8, 23.8, 2.4, and 0.9 days. The BOD loadings ranged from 17 to 58 lb BOD/1000 cu ft/day.

A plot of BOD removal efficiency at various hydraulic detention times is given in Fig. 4. The BOD removal efficiencies were all in the range of 79 to 90 percent. The results shown that BOD efficiency is relatively independent of hydraulic detention time above 1 day.
The efficiency is also relatively independent of BOD loading since the loads varied between 17 to 58 lb BOD/1000 cu ft/day. The plot of gas production versus hydraulic detention time in Fig. 5 also indicates that digestion efficiency is independent of hydraulic loading in a baffled tank provided with filter media.

This laboratory study points out the difficulty in attempting to analyze large-scale systems in the laboratory. Instead of studying a simple anaerobic lagoon, the laboratory system was modified to a baffled lagoon containing an anaerobic filter. Several general conclusions may be inferred from the laboratory data: (1) the concept of biological solids retention time, as documented by several authors,[12] is extremely important in anaerobic processes; (2) an anaerobic filter is very effective in reducing loss of biological solids and preventing passage of raw suspended solids; and (3) if the solids retention time can be increased by changes in design of anaerobic lagoons, *i.e.*, by including baffling and filtration, significant increases in BOD and hydraulic loadings will be possible.

Anaerobic Lagoon Design

Anaerobic lagoons are begin used extensively for first-stage treatment of meat-processing wastewaters from slaughterhouses and packinghouses in rural locations for the following reasons:

1. meat-processing wastes are very amenable to anaerobic treatment;
2. lagoons have low first cost with minimum operation and maintenance costs;
3. anaerobic lagoons can accept highly variable BOD loadings and shock loads that result from accidental spills or unavoidable dumping of grease or blood, without significant adverse effects;
4. anaerobic lagoons can be loaded intermittently, *e.g.*, no loads on weekends, without loss of efficiency; and,
5. anaerobic lagoons are simple systems that require very little operational control.

The disadvantages of anaerobic lagoons are:

1. anaerobic lagoons must be followed by secondary aerobic treatment because of incomplete anaerobic metabolism; and,
2. in the case of urban locations, anaerobic lagoons requre significant land area and may emit odors.

Pretreatment of Wastewater. If anaerobic lagoons are used for first-stage treatment, it is essential that paunch manure and heavy settleable solids be removed from the wastewater to reduce solids accumulation in the lagoons. This is best achieved by segregating paunch contents for separate disposal and treating all process wastewater in a settling-skimming tank. Grease removal should be sufficient to prevent massive accumulations of grease in the lagoons, but it must not be so efficient that an adequate grease cover will not form on the liquid surface. Blood and other by-products should be recovered from wastes for economical operation. The volume of wastewater should be reduced as much as possible to reduce daily flow and retain a high wastewater temperature.

Dimensions. Anaerobic lagoons should be constructed as deep as possible with minimum surface area. The liquid depth should be 15 ft or greater and preferably not less than 12 ft. Side slopes should be as steep as possible for soil conditions at the site. In actual construction, side slopes have been as steep as 1 to 1 and as flat as 3 to 1. Where site soil conditions are suitable for lagoon construction, natural cut and compacted fill embankments of 1½ horizontal to 1 vertical should provide stable interior slopes (Fig. 6). The outside dike slopes should not be steeper than 3 to 1 to facilitate grass mowing. Anaerobic lagoons are generally constructed rectangular with the length approximate twice the width.

Inlets and Outlets. Inlets should be placed near the bottom so that influent wastewater mixes with the active sludge solids which settle toward the bottom. However, they should not be placed directly on the bottom where heavy settleable solids may accumulate. An inlet invert elevation of 2 to 3 ft above the lagoon bottom has proven to be satisfactory. Although multiple inlets are used in large lagoons, a single inlet for a lagoon bottom width of 20 ft to 30 ft is generally satisfactory.

Outlets should be placed below the liquid surface submerged sufficiently to prevent clogging by floating grease. An outlet invert elevation 2 ft below the liquid surface is normally satisfactory. Multiple outlets are recommended to reduce exit velocities which tend to disturb the sludge blanket resulting in loss of active sludge solids thus reducing solids retention.

Inlet and outlet structures must be designed for easy maintenance. Grease in the influent wastewater may cause periodic clogging of inlet pipes, therefore, it is recommended that inlet pipes be 12-in. diameter with provisions for easy cleanout. An anaerobic lagoon effluent is very corrosive. Splitter and outlet boxes should be covered with corrosion resistant open grating, and fiberglas, rather than metal, stops and guides are recommended.

Recirculation Pumping. Several anaerobic lagoon installations have recirculation systems. The most popular system pumps sludge from the bottom near the effluent end of the lagoon back to the inlet box. In two-stage anaerobic lagoon systems, recirculation is conventionally provided from the bottom of the second-stage lagoon to the inlet of the first-stage lagoon, in addition, recirculation for mixing within the first-stage lagoon is provided.

Based on available published data and personal observation, recirculation within a lagoon does not improve removal efficiencies at current lagoon loading rates. Apparently gasification, and inlet flow entering below the surface of the sludge blanket, provides adequate mixing.

Multiple Cells. It is recommended that anaerobic lagoon installations be constructed with a minimum of two cells designed for parallel operation. The primary reason for parallel construction is to permit operation with one cell temporarily out of service.

Few published reports have evaluated two-stage anaerobic lagoons.[8] The concept of returning settled sludge from a second-stage cell to a first-stage lagoon to increase the solids retention time may have some merit, but two-stage systems have not proven to be more efficient than single-stage cells having the same overall detention. It is difficult to maintain an adequate grease cover on a second-stage lagoon, therefore, heat losses from secondary cells can result in lower operating temperatures. Based on the limited published information and personal observation of two-stage and single-stage lagoon systems, there appears to be no advantage in constructing anaerobic lagoons in series. Single-stage lagoons, without provisions for recirculation, appear to be the optimum system for both BOD removal efficiency at current lagoon loading rates and minimum cost of construction.

Solids Retention and Accumulation. Maximum solids retention time can be achieved by constructing deep lagoons with steep sides and proper placement of inlets and outlets. A field study[6] that included sampling the interior of an anaerobic lagoon revealed that two district layers formed within a lagoon. The lagoon had a liquid depth of 15 ft and cell walls constructed with 1½ to 1 side slopes. The upper layer, 9 ft in depth, consisted of supernatant containing approximately 0.1 percent volatile solids. A well-defined surface of a sludge blanket existed 6 ft above the lagoon bottom. The bottom sludge layer consisted of a flocculant, black, digesting sludge containing 3 to 4 percent volatile solids (Fig. 2). This dense sludge blanket occupied approximately one-third of the liquid volume of the lagoon.

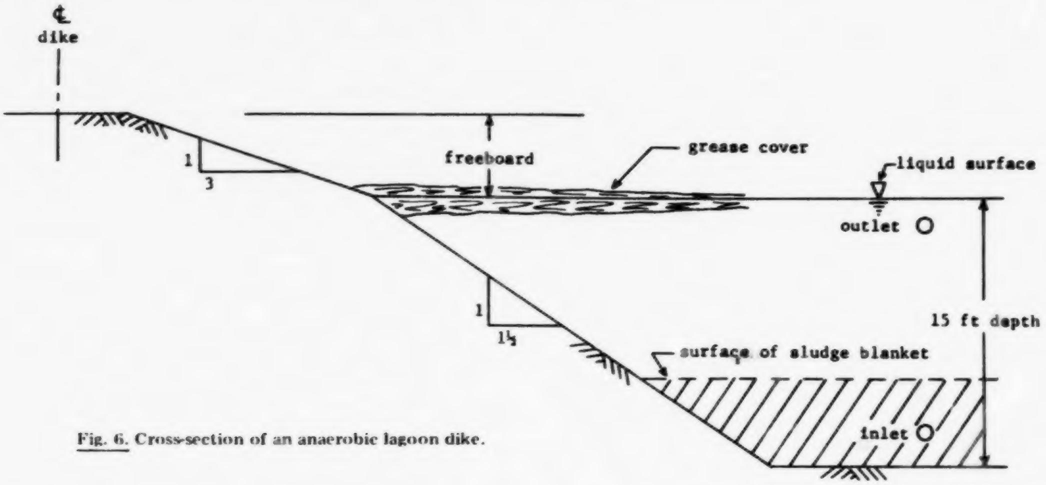


Fig. 6. Cross-section of an anaerobic lagoon dike.

The volatile solids retention time, calculated by dividing lb of volatile solids in the lagoon by lb of volatile solids discharged in the effluent daily, was approximately 100 days. (The BOD removal in this lagoon was 80 to 85 percent under an average BOD loading of 31 lb BOD/1000 cu ft/day.)

The accumulation rate of solids in a lagoon where the wastewater is treated for removal of paunch manure and settled in a skimming tank is negligible. Disposal of paunch manure with wastewater will result in a rapid solids buildup. If heavy settleable solids, other than paunch contents, are not removed from process wastewaters in a skimming-settling tank, accumulation of settled solids of several inches per year on a lagoon bottom could result.

Grease Cover and Temperature. A complete cover of accumulated grease is essential for good digestion to occur in an anaerobic lagoon. The grease cover provides insulation to prevent cooling of liquid in the lagoon; prevents wind mixing of the liquid surface, thus retaining anaerobic conditions; and appears to suppress obnoxious odors.

The temperature of raw meat-processing wastewater is generally 80 to 85°F. Temperature drop due to heat loss in an anaerobic lagoon depends upon the condition of the grease cover, ambient air temperature and wind conditions. During winter months in northern climates the temperature drop between influent and effluent is generally 1 to 2°F per day of detention. For a typical lagoon this would amount to a drop in temperature of 5 to 10°F.

Commonly reported operating temperatures for liquid contents in anaerobic lagoons are between 75 and 80°F. Based on literature data, it appears that operating temperatures in excess of 75°F in lagoons with BOD loadings up to 30 lb BOD/1000 cu ft/day do not significantly influence removal efficiencies. At an operating temperature of 70°F or lower BOD removal efficiencies appear to be adversely affected.

Odors. Lagoons emit a characteristic odor, however, except in the immediate vicinity of the inlet and outlet boxes, a well-operating anaerobic lagoon does not create serious odor problems. In installations where secondary treatment is provided by facultative lagoons, these lagoons are more likely to be the source of obnoxious odors.

A high sulfate ion concentration in process water can cause serious hydrogen sulfide odors when the wastewater is treated anaerobically. The authors are familiar with one extremely foul smelling anaerobic lagoon system where excessive hydrogen sulfide odor is the result of a sulfate concentration greater than 500 mg/1 in the water supply. The anaerobic lagoon effluent contains a hydrogen sulfide concentration in excess of 100 mg/1.

Loadings and Removal Efficiencies BOD loadings in operating lagoons range from less than 10 to greater than 30 lb BOD/1000 cu ft/day, while detention times generally range from 4 to 10 days. Common design criterion specified by regulatory agencies is 15 lb BOD/1000 cu ft/day. For a wastewater BOD concentration of 1400 mg/1, a 15 lb BOD design loading would result in a detention time of 5.8 days.

BOD removal efficiencies range from 60 to 90 percent, however, there is no consistant correlation between removal efficiencies and BOD loadings. Low BOD loadings do not necessarily mean high BOD removal efficiencies, nor do high loadings relate to low removal efficiencies. Furthermore, there does not appear to be any correlation between removal efficiencies and detention times. Although solids retention time is probably one of the most significant factors affecting removal efficiency, there are few data available to evaluate this parameter. The common removal efficiency allowed by regulatory agencies is 60 percent BOD removal in anaerobic lagoons loaded at 15 lb BOD/1000 cu ft/day.

After receiving available anaerobic lagoon loading and efficiency data, it appears that those lagoons loaded in excess of 15 lb BOD/1000 cu ft/day operated consistantly at BOD removal efficiencies between 75 and 85 percent provided that the lagoon was a single-stage system operating at a temperature of 75°F or higher. Based on these findings, the authors recommend the following design criteria for properly constructed single-stage anaerobic lagoons: BOD loadings of 20 lb BOD/1000 cu ft/day; minimum detention time of 4 days; BOD removal efficiency of 75 percent; and minimum operating temperature of 75°F.

Summary

1. BOD and wastewater flow per unit of meat production varies significantly from one meat-processing plant to another. Furthermore, the daily wasteload from any given plant can vary considerably from the average daily wasteload depending upon the quantities of products processed and the operation of by-product recovery units. The 5-day work week and the operational period of 12 to 18 hr per weekday creates a wasteload that fluctuates between day and night, and between weekday and weekend.
2. Meat-processing wastewater due to its high strength, adequate inorganic nutrient content, and relatively high temperature is amenable to anaerobic treatment.
3. Anaerobic processes currently being used to treat meat-processing wastewaters are the anaerobic contact process and anaerobic lagoons. Anaerobic lagoons are preferred for the first-stage treatment of meat-processing wastewater in rural locations.
4. The following factors must be considered in design of anaerobic lagoons: pretreatment of the wastewater; lagoon dimensions, particularly liquid depth and side slopes; and placement of inlets and outlets.
5. Recommended design criteria for single-stage anaerobic lagoons are: average BOD loading of 20 lb BOD/1000 cu ft/day, minimum detention time of 4 days, BOD removal efficiency of 75 percent, and minimum operating temperature of 75°F.

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ANAEROBIC LAGOON TREATMENT OF PACKING WASTES IN LOUISIANA

by

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Within the last ten years, nearly fifty abattoirs and packinghouses in Louisiana have installed anaerobic ponds to treat wastes from their operations.[1] Use of anaerobic ponds for this purpose has been actively promoted by the Louisiana State Department of Health after observing satisfactory performance of such a facility at the Autin Packing Company, Houma, Louisiana. Anaerobic ponds have found favor with the Louisiana meat packers because of low installation cost, ability to handle "shock loads", and dependable nuisance-free performance.

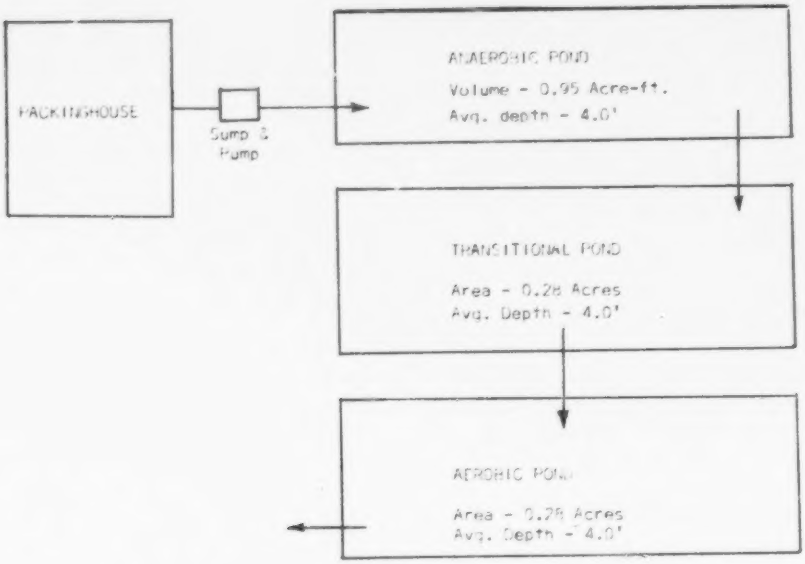

Figure 1, Autin Packing Co., Houma, La. Flow diagram of waste disposal facilities.

In the typical packinghouse installation, all of the wastes from slaughtering, including paunch manure, blood, fleshings, and other wastes not profitably salvaged, and sanitary wastes incidental to plant operation are discharged directly into the anaerobic pond. The effluent of these anaerobic ponds usually receives additional treatment in one or more aerobic ponds before discharge to receiving streams.

History:

The anaerobic and secondary ponds at the Autin Packing Company in Houma were installed sometime in 1948 according to the recollection of the owner, Mr. A.A. Autin. However, there are no records available to definitely establish the date of original use. Mr. Autin also indicated that the inner levees separating the series of three ponds (see Figure #1) originally contained French drains made of oyster shells. Some treatment was apparently expected as the waste water trickled through the oyster shells between the three ponds in series. The oyster shell French drains soon clogged, and trenches were cut across the inner levees to allow the liquid to pass through the series of ponds to the effluent drainage ditch. The system operated without nuisance and the effluent from the system caused no problems in the receiving ditch running by several residences on adjoining properties. Little attention was paid to this operation for many years.

Interest in the Autin disposal system developed in May of 1960 when the owners requested advice on improving the existing operation. At that time, the anaerobic and the first of two secondary ponds were almost filled with sludge and repairs were needed on the levees. After dredging to remove excess sludge, and repair of the levees, study and evaluation of the Autin disposal facilities began with the idea of developing design criteria for the use of similar facilities at other packinghouses.

The three ponds at the Autin plant in Houma are approximately the same shape and size. The average liquid depth in the ponds was approximately 2 feet in May, 1960 but was deepedned to about four feet when the ponds were reconditioned in the fall of 1960. Figure No. 1 shows dimensions and arrangements of facilities at this installation.

All of the waste from the plant, with the exception of the drainage from the hide house, drains into a large sump from which it is pumped to the anaerobic pond by a heavy duty sewage and trash pump. As some difficulty was experienced at the outset with solids deposition in the sump, due primarily to paunch manure, a large stirring device was installed in the sump to mix the waste into a slurry that could be handled readily by the pump.

The high solids content raw waste caused the shallow anaerobic pond to fill with sludge. Mr. Autin had these accumulations removed from the pond with a dragline and stacked into large piles in the yard adjacent thereto. The sludge thus deposited has not produced an odor or insect problem and most of the dried accumulation in the yard was removed by truck farmers for use as a soil conditioner. Until the pond was deepened in 1960, the sludge has been removed approximately biennially.

At the time the anaerobic pond at Autin was observed in May 1961, its surface was covered with a thick scum or crust. It was assumed that formation of the surface crust was essential to the odor free operation of the pond.[2] However, anaerobic ponds installed subsequently at other packinghouses sometimes did not become "crusted over" but nevertheless operated without odor nuisance, probably due to an anaerobic environment favorable for the predominance of methane fermentation.[3] Paunch manure was observed to be the most prominent ingredient of the crusts that have formed.

Design Criteria:

Shortly after tests and observations began at Houma, it became apparent that the anaerobic pond was a very effective waste treatment device. The problem then was to determine how this knowledge could most effectively be put to use elsewhere. Several meat packers were already interested in using similar facilities because of the low installation cost in comparison to other treatment methods and the ability of the pond system to handle troublesome paunch manure. Accordingly, design criteria was developed on the basis of the results obtained at the Houma installation.

Inasmuch as data on raw waste volume and strength at the Houma installation was not available in early 1961, the decision was made to comparatively size disposal facilities at future installations on the basis of the number and type of animals slaughtered in a representative time period.[4]

Slaughtering load was calculated in terms of the total waste from slaughtering one hog, hereafter referred to as one "hog unit". Hog unit equivalents for the waste from slaughtering animals other than hogs are estimated according to the following table:

ANIMALS SLAUGHTERED	*WASTE EQUIVALENT*
One hog	1 hog unit
One sheep	1 hog unit
One calf (less than 600 lbs. live weight)	1 hog unit
One medium beef (600 to 900 lbs. live weight)	3 hog units
One heavy beef (over 900 lbs live weight)	5 hog units

The "hog unit" method of estimating waste was selected because most abattoir and packinghouse operators in Louisiana can estimate accurately how many animals will be processed but have no means of estimating plant waste volume or strength. Estimates of the number of animals to be killed per week was adopted as the most convenient planning unit.

The Autin Packing Company furnished detailed information on the number, type, and size of animal slaughtered each month during calendar year 1960. Computation by the hog-unit method indicated an average weekly loading of 386 hog units.

Since waste loadings on anaerobic ponds are normally calculated in terms of volume, the loading on the Houma anaerobic pond before deepening was calculated at approximately 800 hog units per week per acre-foot and 400 hog units per week per acre-foot after deepening. 500 hog-units per week per acre-foot was arbitrarily selected as the design criteria for future anaerobic pond installations.

While the anaerobic pond at Houma was originally very shallow (2 feet deep) it was recommended future installations be made as deep as possible to reduce area requirements and aid in early "crust" formation. Deeper ponds have subsequently been found to perform satisfactorily, as expected. An easy access along one side for cleaning sludge accumulations was also recommended.

Evaluation Procedures:

Following adoption of standard design criteria by the State Department of Health, several meat packinghouses built anaerobic pond installations. Two of these installations, one at Slidell, La. and another at Gonzales, La., were closely observed and evaluated along with the Houma installation, and the results are included in this report.

The Slidell installation (Guillot Packing Company) is owned and operated by Mr. Arthur J. Guillot. Operation of the plant and waste treatment facilities began in April of 1961. A flow diagram for these facilities is shown on Figure No. 2. These waste stabilization ponds were designed by Lionel E. Flotte and Associates, Consulting Engineers, New Orleans, Louisiana using the design criteria cited previously.

Operation at the Slidell facility is similar to the operation at Houma. All of the waste from slaughtering, including blood, paunch manure and sanitary wastes are discharged into a hopper bottomed receiving sump and pumped from there to the anaerobic pond. The top of the anaerobic pond covered over with scum in February of 1962.

Information on the total number of animals killed over a six month period indicates a weekly killing load at the Slidell installation of 600 hog-units, and an anaerobic pond loading of 667 hog units per week per acre-foot.

The Gonzales installation (Stevens Meat Company) is owned and operated by Mr. Leo Stevens. Operation of the treatment facilities began in January, 1962. A flow diagram for these facilities is shown on Figure No. 3. The pond system was designed by the author and built by the owner.

This system has operated satisfactorily and without nuisance from the outset. The ponds remedied a very serious stream pollution problem.

The average weekly killing load at the Gonzales plant is 1,328 hog-units, making the average weekly loading on the anaerobic pond 332 hog-units per acre foot.

As no funds or personnel were available for a special study and evaluation project, a limited study and evaluation was worked into the existing field service program of the State Department of Health. This was to be accomplished by scheduling occasional visits to these installations by field engineers for the purposes of making observations and collecting samples for analysis. Although determinations for B.O.D., settleable solids, suspended solids total solids, volatile solids and pH were made in the course of the investigation, this report is concerned primarily with B.O.D. reduction. Analysis procedures were according to the latest edition of "Standard Methods for the Examination of Water and Wastewaters".[5]

At the time of the visits to these waste disposal facilities, the engineers would observe the physical condition and appearance of the ponds, pond effluent and receiving ditch or stream, and submit a detailed report. If slaughtering operations were in progress on the days of the visit, samples of the raw waste were collected for B.O.D. analysis. Samples for B.O.D. analysis from the ponds were collected at a point about 6 inches below the water surface near pond outlets.

At the Houma and Gonzales plants included in this study, private wells are used as the primary source of water in the plant operation. The Slidell plant has unmetered use of the City supply. Water consumption figures were therefore not available for use in estimating waste flow volume. In March, 1961, the waste pump flow rate at the Houma installation was calibrated and a time lapse meter installed in the electrical circuit of the pump to record pump operation. Waste flow from the Houma installation was found to average 214 gallons per hog-unit. Flow measurements at four other Louisiana packinghouses not covered in this report were found to average 135, 200, 245

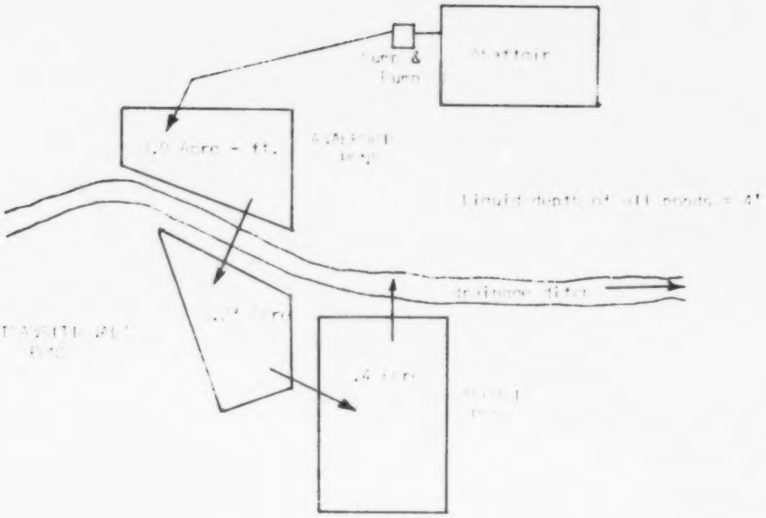
Sump &
Pump
Abattoir
1.0 Acre - ft.
Liquid depth of all ponds = 4'
drainage ditch
.4 Acre

Figure 2. Guillot Packing Company, Slidell, Louisiana flow diagram of waste disposal facilities.

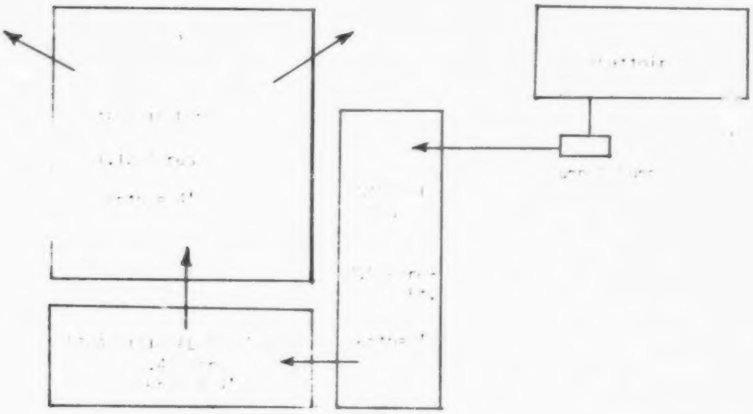

Figure 3. Stevens Meat Company, Gonzales, Louisiana Flow diagram of waste disposal facilities.

Table 1. Autin Packing Co. waste treatment facilities Houme, La. Results of BOD analysis.

Date	Composite Raw Waste ppm BOD	Anaerobic Pond Effluent ppm BOD	% BOD Removal
December 13, 1960		340	
January 31, 1961	3,900		
February 16, 1961		252	
February 23, 1961		300	
March 1, 1961		186	
April 5, 1961		213	
April 13, 1961	2,600	161	93.8
June 15, 1961	2,723	166	94.0
July 6, 1961	1,250	69	94.4
October 5, 1961	2,100	63	97.0
December 19, 1961	2,800	36	98.7
January 25, 1962	920	158	82.8
MEAN	2,328	177	92.4

Table 2. Guillot Packing Company waste treatment facilities Slidell Louisiana, results of BOD analysis

DATE	Composite Raw Waste BOD ppm	Anaerobic Pond Effluent BOD ppm	% BOD Removal
September 21, 1961	1,750	-	-
September 28, 1961	1,480	-	-
October 12, 1961	2,080	565	72.8
October 25, 1961	2,593	462	82.3
November 9, 1961	4,120	602	85.4
January 5, 1962	-	398	-
January 18, 1962	[illegible]	131	95.1
March 26, 1964	785	169	78.5
April 3, 1964	465	131	71.8
April 17, 1964	1,306	(sludge)	-
February 9, 1967	842	471	44.1
February 17, 1967	1,215	600	50.6
February 22, 1967	2,910	506	82.6
April 3, 1970	706	196	72.2
MEAN	1,765	412	76.7

and 20 gallons per hog-unit. The 20 gallon figure came from a small rural establishment killing a small number of animals weekly, and dry collecting most wastes.

Results:

The results of B.O.D. analysis of untreated and treated packinghouse waste at each of the three anaerobic pond treatment facilities described in this report are shown in Tables No. 1, No. 2, and No. 3. These indicate an average B.O.D. reduction ranging from 94.3% through a pond receiving wastes from slaughtering 332 "hog units" per week per acre-ft. to 76.7% reduction through a pond loaded at 667 hog units per week per acre-ft.

Figure 4 shows a graph of the percent B.O.D. reduction in each of the three anaerobic pond systems studied versus the waste loading applied to the pond each week in terms of hog units per acre-ft.

Based on the measured average waste flow of 214 gallongs per hog unit at the Autin installation in Houma, Louisiana and the average rawwasteB.O.D. there of 2,328 ppm, the daily B.O.D. loading on anaerobic ponds at the recommended design loading of 500 hog units per week per acre-ft. is approximately 300 pounds of B.O.D. per acre-ft. per day.

The mean annual temperature in south Louisiana where the three study pond systems are located is 68°F. Winters are usually rather mild. Since anaerobic ponds for treating packinghouse wastes can be made rather deep, and "crusts" usually form rapidly over them if paunch manure is received, and such a crust may be of value as an insulator, the anaerobic ponds should perform satisfactorily without artificial heating in somewhat colder climates than prevalent in south Louisiana. Such ponds located in the coldernorthern part of Louisiana have operated without problems but performance data on these to date is very limited.

No odor nuisance, insect nuisance or other problems have been reported to date at any of the 50 such installations in Louisiana except for slight odors at initial start-up.

Summary:

The results of treating packinghouse wastes in an anaerobic pond at Houma, Louisiana, and comparatively designed installations at Slidell and Gonzales, Louisiana, indicate that packinghouse wastes, including blood and paunch manure, can be successfully treated in low cost anaerobic ponds without nuisance or health hazard. These ponds receive an approximate B.O.D. loading of 300 pounds per acre-ft. per day; however, anaerobic pond designs in Louisiana are usually based on the number of animals processed in a given time period (500 hog units per week per acre-ft.)

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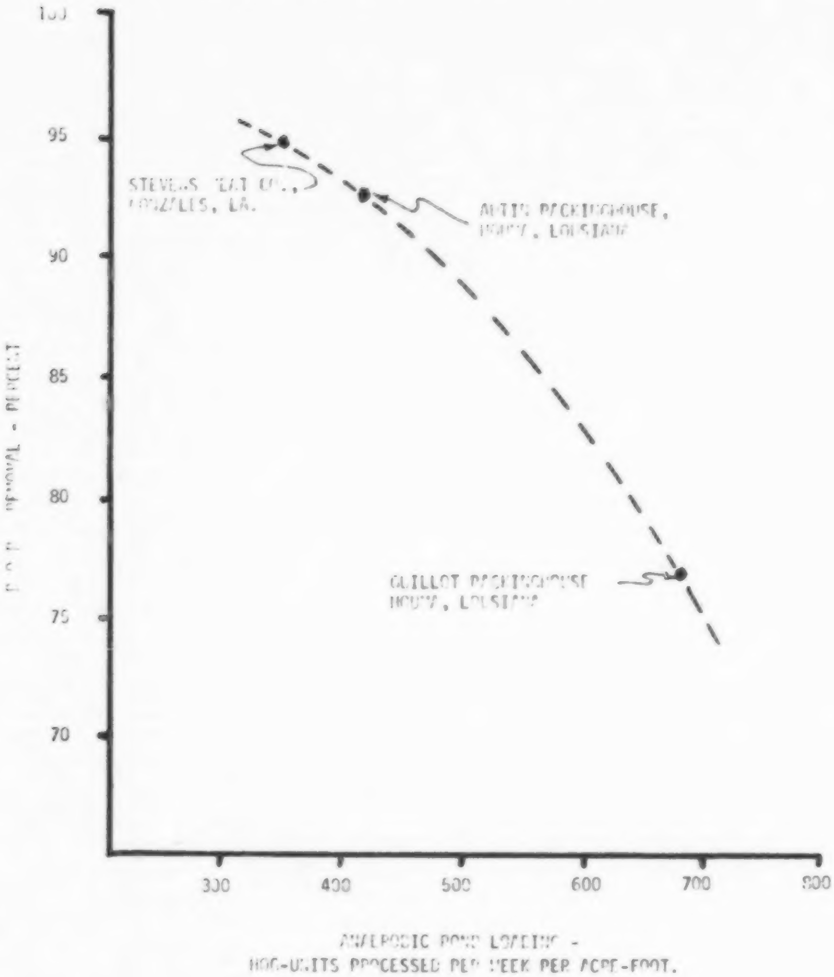
Table 3. Stevens Meat Company waste treatment facilities Gonzales, Lousiana results of BOD analysis

Date	Composite Raw waste BOD ppm	Anaerobic Pond Effluent BOD ppm	% BOD Removal
October 26, 1962	792	132	83.4
August 22, 1963	2,245	140	93.8
February 21, 1964	-	175	-
March 20, 1964	2,545	115	95.5
April 10, 1964	-	87	-
February 10, 1967	2,970	147	95.1
February 16, 1967	1,900	96	94.9
February 23, 1967	2,265	111	95.1
April 8, 1970	986	64	93.5
MEAN	1,957	115	94.3

Figure 4. Packinghouse waste treatment in anaerobic ponds. BOD removal vs pond loading



CURRENT DESIGN CRITERIA FOR ANAEROBIC LAGOONS

James E. White

I. Introduction:

Anaerobic lagoons have played a major role in the treatment of wastes containing high suspended solids and organic concentrations. These lagoons have been compared to uncovered digesters. Current design criteria has, quite often, been based on operating experiences of earlier lagoon installations.

Increasing interest is being placed on anaerobic lagoons due to the strict water quality standards imposed by the various government water pollution control agencies. Anaerobic lagoons have been attempted at very wide range of loading rates, at depths of 1½ ft. to 20 ft., and under all types of weather conditions.

The purpose of this paper is to present current design criteria presently accepted by various government agencies and operation data of existing anaerobic lagoon installations.

II. State Agencies Design Criteria:

Several state water pollution control agencies were asked to submit criteria they have used in their review of anaerobic lagoon projects. None of the states contacted have published design criteria for anaerobic lagoons. However, several of the states did submit criteria they have used in the past in their review of anaerobic lagoon projects.

A summary of the criteria is given in Table 1.

As Table 1 indicates, the major parameter for lagoon design is BOD loading. The average loading for the various states is 12-15 lbs. BOD/1000 ft^3. Expected efficiency was fairly consistent, 60-80% reduction, among the states.

Only one state, Georgia, allowed an increased loading for recirculation. In general the recirculation ratio recommended by Georgia is 1:1.

All states agree that the lagoons should be as deep as possible, 10 to 15 feet.

The lagoons have presently been approved for the following types of wastes:

- Slaughterhouses
  - beef
  - pork
  - poultry
- Feedlots
  - beef
  - hogs
  - poultry
- Rendering Plants
- Textile Dye Plants
- Canning
- Tanneries

III. Loading Characteristics of Existing Installations:

There are numerous articles recorded in the various technical journals on anaerobic lagoons. Unfortunately, many of them contain incomplete data to accurately analyze the lagoon design criteria and lagoon performance. There are also a variety of dimensions to express lagoon loadings. For example, lagoon loading rates have been expressed in the following terms:

lbs. BOD/acre
lbs. BOD/1000 ft^3
lbs. BOD/lb. volatile solids in accumulated sludge
lbs. Volatile solids/lb volatile solids in
accumulated sludge
lbs. suspended solids/1000 ft^3
sq. ft. lagoon area/animal
lbs. volatile suspended solids/1000 ft^3

If the recorded data is incomplete, it is difficult to compare the various facilities in terms of a common loading dimension.

Sollo[1] obtained considerable data on an anaerobic lagoon used in the treatment facilities of a meat packing house. Pretreatment ahead of the lagoon consisted of primary treatment and grease recovery. Over a 3 year period, the lagoon averaged 85 percent BOD reduction. Sludge recirculation was practiced during the sampling program. One surprising factor, however, was the fact that a 10 ft. depth of sludge accumulated in 2½ years. The sludge was pumped to a field for disposal where it dried readily with no nuisance.

Stanley[2] observed the operation of two anaerobic lagoons operated in series at a meat packing plant in Canada. With a detention of 3 to 5 days, the lagoons averaged 80 percent BOD reduction during one composite survey. It was found that the second lagoon provided little additional treatment.

Rollag and Dornbush[3] also conducted surveys on two anaerobic lagoons operated in series on a meat packing waste. The only pretreatment provided was grease recovery. The lagoon loadings for the first cell were 16.1 lbs BOD/1000 ft^3 and 8.2 lbs suspended solids/1000 ft^3. The first cell removed 58 percent BOD and 77 percent suspended solids. The minimum temperature recorded of the lagoon contents was 76.9°F, indicating good insulation being provided by the grease cover. They determined that little additional reduction occurred in Cell No. 2.

Enders, Hammer, and Weber[4] made a study on an anaerobic lagoon in Nebraska serving a slaughterhouse. The average of 3 composite surveys indicated BOD loadings and efficiency of 31.4 lbs BOD/1000 ft^3 and 87 percent respectively. The loadings, based on a different parameter, were expressed as 0.046 lb BOD/lb. volatile solids in the lagoon sludge and 0.055 lb volatile solids/lb. volatile solids in the lagoon sludge. Based on these parameters, the lagoon was loaded at a rate similar to an anaerobic digester.

An anaerobic lagoon system serving a packing house at Cherokee, Iowa[5] was designed for an average BOD loading of approximately 10 lbs BOD/1000 ft^3. The lagoon is presently removing 85 percent of the applied BOD. Recirculation is available up to a 1:1 ratio. During the winter months the lagoon contents remained above 80°F.

The Iowa State Department of Health[6] made an extensive study of five separate anaerobic lagoon systems serving various packing houses. The BOD loadings ranged from a low 9 lbs/1000 ft^3 to a high of 60 lbs/1000 ft^3. In general, the average loadings were in the 12-15 lbs BOD/1000 ft^3 range. The efficiencies of the lagoons were

Table 1. State agencies anaerobic lagoon criteria

State	BOD, lbs/1000 ft^3	Suspended Solids, lbs/1000 ft^3	Detention, Days	Expected Reduction, %
Colorado	None at this date			
Georgia	3,* 15**	1.25*, 8**	-	60-80
Illinois	15-20	--	5	60
Iowa	12-15	--	5-10	60-80
Montana	2-10	--	10 minimum	70
Nebraska	12-15	--	3-5	75
North Dakota	None at this date			
South Dakota	15	--	--	60
Texas	25-100	100-400	5-30	50-100

* No recirculation provided

** 1:1 recirculation provided

Table 2. Anaerobic lagoons for meat packing wastes.

Author	Type of Waste	Loading Rate	Efficiency
Sollo	-	11.2 lbs BOD/1000 ft^{3*}	85**
Stanley	-	-	80
Kollag	Beef	16.1 lbs BOD/1000 ft^3	58
Enders	Beef	31.4 lbs BOD/1000 ft^3	87
Reckert	Beef & Hogs	10.0 lbs BOD/1000 ft^3	85**
Iowa State Health Dept.	Beef & Hogs	12-15 lbs BOD/1000 ft^{3***}	85
Wymore & White	Hogs	13 lbs BOD/1000 ft^3	65

* Pilot Plant Loading Rate
** Recirculation Provided
*** Average of all plants studied

consistently above 70 percent. Those lagoons operating in the 12-15 lbs BOD/1000 ft^3 range averaged 85 percent removal. The one lagoon with the 60 lbs/1000 ft^3 BOD loading also provided excellent treatment, removing 72 percent of the applied BOD in a 3.4 day detention.

The Iowa Health Department also obtained sludge samples from the anaerobic lagoons for determining sludge accumulation. From their studies they estimated that packing houses discharging high suspended solids will have an accumulation of approximately seven inches per year in the anaerobic lagoons. The sludge averages 55 to 65 percent volatile material. Excellent gas production is evident in all the lagoons as volatile acids run consistently below 200 mg/l.

Wymore and White[7] designed an anaerobic lagoon system for a slaughterhouse in Iowa based on 15 lbs BOD/1000 ft^3/day loading rate. Anticipated removal was 60 percent. The lagoons are presently being loaded at an average rate of 13 lbs BOD/1000 ft^3/day and are providing approximately 65 percent removal during the winter months. The temperature of the lagoon contents during the winter months averaged 60°F.

Table 2 is a summary of the lagoon loading rates discussed.

Several field studies have also been conducted on the disposal of livestock wastes by anaerobic lagoons. Dornbush and Anderson[8] assisted an owner of a mechanized egg production unit who had major odor problems with his anaerobic lagoon. The odor was eliminated by raising the water level in the lagoon which reduced the lagoon loading and by providing intermittent mixing. Lagoon loading rates of 5 to 10 lbs. volatile solids/1000 ft^3/day provided satisfactory results.

Table 3. Anaerobic lagoons for livestock wastes

Author	Type of Waste	Loading Rate	% Removal
Dornbush	Poultry Manure	5-10 lbs VS/1000 ft^3	-
Loehr	Milking Parlor	8.6 lbs BOD/1000 ft^3	85
Bhagat	Dairy Manure	20 lbs BOD/1000 ft^3	88

Table 4. Characterisites of packinghouse and beef feedlot wastes.

	Packinghouse Wastes[7]	Beef Feedlot Wastes[10,11]
BOD: Volatile Solids	0.8 to 0.9	0.3 to 0.5
BOD: COD	0.4 to 0.5	0.3
BOD: Total Solids	0.6	0.2 to 0.3

Loehr[9] performed studies on an anaerobic lagoon treating milking parlor wastes. The lagoon was loaded at about 8.6 lbs BOD/1000 ft^3 during the summer months and provided 85 percent reduction. This loading rate would be approximately one-fifth the loading of a conventional single stage digester. The raw wastes entering this lagoon did not have the characteristics of typical livestock wastes. Evidently the milking parlor floors were dry cleaned before wash down.

Bhagat[10] determined the loading rate on a lagoon treating dairy manure was about 20 lbs BOD/1000 ft^3 per day and provided 88 percent BOD removal. In terms of volatile solids, the loading rate was 70 lbs/1000 ft^3 per day. Thus only about 28 percent of the biological solids are biodegradable according to the 5-day BOD test.

Table 3 is a summary of the loading characteristics of the manure lagoons discussed.

IV Discussion

A Organic Loading:

As presented, the majority of states where anaerobic lagoons are located generally allow loading rates in the range of 12 to 15 lbs BOD/1000 ft^3 per day.

Operating experiences of lagoons treating packing house wastes certainly support this loading range for providing excellent treatment. However, there are also some data available to justify higher loading rates. Two separate anaerobic lagoon installations serving packing houses in Iowa have operated at 20 to 25 lbs BOD/1000 ft^3 per day loading rates and provide 75 to 85 percent BOD removals. The pH of these lagoons remains near 7.0 while the volatile acids remain relatively low, less than 200 mg/l.

The wastes from feedlots have different characteristics than packinghouse wastes. Some of the characteristics are shown in Table 4.

Table 4 clearly indicates the differences between the two wastes. Thus, when one compares the loading rates of lagoons treating different types of wastes, care should be taken in making direct comparisons without knowing the waste characteristics.

It is obvious that an anaerobic lagoon treating a feedlot waste will accumulate considerable more nonbiodegradable material than a lagoon treating a packinghouse waste if both lagoons are loaded at the same rate. For this reason, anaerobic lagoons treating feedlot wastes usually need to be cleaned considerably more often than packinghouse waste lagoons.

Acceptable loading rates for anaerobic lagoons treating feedlot wastes are highly variable. If a lagoon is in an isolated area, where odors are not a problem, quite often the Owner will be able to receive adequate treatment at a higher loading rate than if the lagoon were in a location where odors would be a nuisance. Another important factor to be considered when attempting to arrive at a lagoon loading rate is how often the owner wishes to remove the accumulated solids from the lagoon. High loading rates will obviously cause a lagoon to become filled with solids sooner than one at a low loading rate.

In general, lagoons treating feedlot wastes have demonstrated satisfactory results at loading rates of 10 to 20 lbs BOD/1000 ft^3 per day. Considerably higher loading rates have also provided satisfactory treatment, but have the major disadvantage of requiring sludge removal more often.

B. Temperature:

The temperature of the lagoon contents plays a major role in the operating efficiency of the anaerobic lagoon. Research shows that lagoons which continue to operate satisfactorily during winter months usually have a lagoon temperature above 75°F. These lagoons always have an excellent cover consisting of grease and inert solids. If the lagoon temperature drops below 55 or 60°F, gas production usually drops to a minimum. When this happens, the lagoon does little more than act as a settling basin.

C. Lagoon Depth:

It is common knowledge that an anaerobic lagoon functions much better when deep, 10-15 ft., than when shallow. Wind action has a much less effect on the lagoon contents and the scum layer. Usually it is more economical to construct a deep lagoon than a shallow one because of the land areas required.

However, care should be taken not to place the lagoon bottom near groundwater. With the present water quality standards now in force, one does not wish to contaminate a ground water supply. Also, the temperature of the ground water will certainly lower the temperature of the sludge on the bottom of the lagoon. Since this is where a major portion of the gasification occurs, if the temperature drops below 55 to 60°F, the efficiency of solids and BOD reduction in the lagoon will certainly be reduced.

D. Recirculation:

In theory, sludge recirculation, provides definite advantages and increased efficiency in lagoon operation. Any type of biological treatment is greatly improved when the active microorganisms are brought in immediate contact with the raw wastes. This has been clearly proven in high rate anaerobic digesters and activated sludge processes.

However, there has been little research in anaerobic lagoon installations to determine the exact design parpmeters for optimum recirculation rates. Economics is certainly a major factor in determining the merits of recirculation. For example, if 1:1 recirculation allowed loading rates to be increased by 20 percent, the construction and operating costs of the recirculating pump station must be compared with the construction cost of increasing the lagoon volume by 20 percent. The numbers in this example are not based on any installation, but are used only to illustrate the importance of an economic study of a proposed lagoon. However, until more field data is obtained, little can be said considering design requirements for sludge recirculation.

E. Series vs. Parallel Design:

There seems to be little advantage in operating anaerobic lagoons in series. Those installations which have been operated in series show very little increase in efficiency than those operated in parallel. However, if high recirculation rates are provided, a second cell in series may be warranted to provide a more clarified effluent.

F. Effluent Quality:

The quality of the effluent from an anaerobic lagoon usually needs additional treatment before it can be discharged to a water course. The BOD may range from 150 mg/l on up, depending on the lagoon loading rate and other factors previously discussed. The effluent [illegible] has no dissolved oxygen.

Since there is little nutrient removal in anaerobic lagoons, the majority of nutrients in the raw waste are discharged in the effluent.

V. Conclusions:

Anaerobic lagoons will provide excellent treatment of wastes containing high organic and solids concentrations when designed and operated properly. Most experience with anaerobic lagoons has been in the meat packing and feedlot industries. However, anaerobic lagoons have also served the canning, textile, rendering, and tanning industries.

The economics of construction and operation of anaerobic lagoons certainly make them appealing for the various wastes suitable for this type of treatment.

Although efficiencies are usually in the 80 percent or above range, the effluent requires further treatment before being discharged to receiving streams.

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CHALLENGE FOR WASTE WATER LAGOONS

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My purpose here today is neither to praise lagoons nor to lay them to rest, but to issue a challenge to lagoon technology. In view of the more stringent water quality standards now emerging, do lagoons really have a future in wastewater treatment? Community aesthetics, population trends, and environmental constraints tend to indicate that lagoons might not be a treatment process of the future. In general lagoons suffer from the lack of positive control. Lagoon performance is greatly affected by environmental conditions such as temperature, solar radiation, and wind speed. These factors are not easily controlled on a daily basis. While lagoons are simple to construct, the reactions bringing about purification are complex.

Properly designed and well operated lagoons have provided a certain degree of purification to many communities at a cost comparable to individual household units with greater benefit to the homeowner. However, like other wastewater treatment devices, they have advantages and disadvantages and should not be considered as a panacea for all sewage treatment needs. Like any treatment unit, they are subject to overload and, like any treatment unit, when overloaded their effectiveness can be expected to suffer. Like any biological treatment unit, continual change and variation in performance appears to be normal behavior rather than the unusual or abnormal. There tends to be a gradual deterioration in the effluent quality as lagoons begin to age.

On the basis of loading commonly employed, it is apparent that required areas at least for the waste stabilization pond preclude the economic use of such facilities in many localities. Severe odor problems can occur at the time of ice breakup in the spring. While serious nuisances may be only for a few days, the periods are long enough to create many complaints. For this reason lagoons must be placed away from habitation. Odors can travel long distances. Aesthetic failures are thus common where lagoons are used.

Groundwater pollution from a lagoon must always be regarded as a possiblity. If a lagoon is built in an area having a soil of high porosity or unsound geological formation, the danger of groundwater pollution becomes real if steps are not taken to seal the lagoon. Where seepage is directly into an aquifer, bacteria and other pollutants can travel great distances. Plastic sheeting for sealing lagoons is not without problems. The sheeting is easily ruptured and subject to bulging if soil underneath contains organic material which will produce gas under anaerobic conditions.

If I continue to raise questions concerning the utility of lagoons, it will sound as if we have no interest in them. We in the Federal Government have had and do have an interest in lagoons as a method of treatment. Our construction grants program has supported the building of many of the waste stabilization lagoons now in use. From 1960 to 1968 the number of lagoons increased from 1350 to 3450. The population served by lagoons went from 2.2 million to 6.1 million in the same period of time.

We do feel, however, that now is the time for decision. Much effort and money will be spent in the next few years on treatment facilities and we must be certain that properly designed lagoons can restore wastewater to a quality equal to or better than effluent from secondary treatment. *Will lagoon treatment of wastes meet water quality standards and are lagoons equivalent to secondary treatment? This is the challenge lagoons face today.* For our discussion we will adopt the following liberal definition of secondary treatment:* Scondary treatment is any treatment given wastewater that will produce an effluent containing less than 30, 75, 25 mg/l of BOD_5, COD, and suspended solids, respectively. These values shall not be exceeded more than 10% of the time on daily composite samples taken proportional to flow. The effluent produced shall be nontoxic and free of offensive color and odor. In addition, the effluent must meet the bacteriological quality standards promulgated by the various states.

A proposed rule dated June 10, 1970 to govern grants for construction of treatment works calls for: 1) substantially complete removal of all flotable and settleable materials, and 2) 85% removal of BOD demand based on a monthly average. Under some conditions the smaller community may get exempted from these requirements but the trend for all treatment is clearly up.

What are some of the difficulties that lagoons may encounter in meeting these requirements?

Oxygen Demand Removal

Rational approaches to design loading for lagoons have been developed, but due to many uncontrollable factors, no approach has been universally accepted. The loading of organic matter on a lagoon is a function of the rate at which the biological processes stabilize it without any nuisance. Therefore, the allowable organic load is controlled by all such biological removal processes. Temperature plays a very important role in operation of lagoons. At low temperature, the amount of BOD removal is poor. When long periods of ice cover occur, it is impossible to maintain aerobic conditions, even at very low loading, unless aeration is supplied. In certain parts of the country the lagoons may be sealed for 40% of the year. Therefore, factors governing the maintenance of an aerobic condition under open water surface no longer apply. The presence of ice with an overlaying layer of snow limits the

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*** This definition is for discussion only in this paper and does not represent FWQA policy or rules**

amount of light energy that reaches the water. The reduction usually ranges between 90 and 96%. There is generally no measurable oxygen production during winter under ice and, consequently, anaerobic conditions are continuous from the onset of ice cover to ice breakup. If effluents are drawn during this time, they are void of oxygen and not acceptable to the stream.

Effluents from lagoons can contain a large amount of algal cells and other biological matter which when discharged to a river can result in secondary pollution if the algae die. Many feel that this is not a problem; however, it must be recognized that algae when they die release organic compounds into the stream. One mg/l of dead algae is equivalent to about 1.5 mg/l of 5-day BOD. With algae concentration in lagoon effluents ranging from 50 to 125 mg/l, or more, the BOD load discharged to the receiving stream would be in the order of 75 to 150, or higher. Algae are a source of pollution just as the organics present in the original wastes. Are algae a better form of organic matter than that originally present in domestic wastewater? Some of the algae may be used as food by fish food organisms, but most of the organisms settle to the bottom and decompose. Decomposition of dead algae causes the same adverse effects as do other kinds of organic wastes. A lagoon may produce one pound of algae for every pound of BOD fed. Unless the algae are removed or digested anaerobically, only the form of the organic matter has been changed by the treatment.

In aerated lagoons, the turbidity and turbulence prevents normal algae growth. The key to effluent quality in the aerated lagoons lies in the removal of the suspended solids. These are the major source of BOD in the effluent. Without solids removal the BOD removal will be less than satisfactory. How do you take these solids out of the system? This is another challenge to lagoons. If leakproof lagoons are designed and constructed for no discharge, this is indeed a powerful pollution control method. Such a system at some time will have to have solids removed but if large areas of cheap land are available, it may be a matter of years before any thing has to be done.

In evaluating the literature on lagoons and their ability to remove oxygen-demanding materials, it is not always clear whether the results are based on filtered or unfiltered samples. In many cases the results are given only on filtered samples, leaving the reader to guess what the total oxygen demand might have been on the receiving stream. Many of the studies were of such short duration that seasonal variations were not included. The practice of reporting lagoon performance over short periods after start-up or after major modification does not represent the performance found once equilibrium conditions are obtained. During the early life of a stabilization lagoon, a portion of the observed reduction of the BOD in the lagoon is due to settlement of organic material as sludge. Anaerobic fermentation reduces the sludge but releases products of fermentation to the liquid layer. Under uniform conditions, a state of equilibrium will be reached in which the BOD deposited as sludge is equal to the BOD released in the decomposition of the sludge. Once equilibrium conditions are obtained there is an annual cyclic variation of sludge accumulation due to changes in temperature between winter and summer and vice versa.

In any wastewater treatment program it is important to have methods for measuring the effectiveness of treatment and for assessing the polluting effect of the treatment plant effluent discharging to a river. The trends would dictate that in the near future the total polluting effects of a waste treatment plant effluent will be considered. Not only will the short-term oxygen demand be considered, but also the ultimate oxygen demand will be appraised. It is sometimes shocking to find that removal expressed as BOD_5 may be only one-fourth of the long-term oxygen demand. *This will mean an effluent low in biological solids. Will lagoons be able to meet this challenge?*

Solids Removal

A high concentration of suspended solids in lagoon effluent is one of the major disadvantages. Suspended solids in lagoon effluents follow somewhat the same variations as does the BOD. The preceeding section on oxygen demand removal covered to a certain extent some of the problems. They will not be repeated here except to say that the presence of dense algal concentrations and other biological solids in lagoon effluents in many instances renders the effluent unsuitable for reuse or discharge in a receiving stream.

Many methods for algae removal have been studied, but none were found to be economical. Some of the methods tried were microstraining, flotation and chemical coagulation. Algae removal should be differentiated from algae harvesting where optimization of conditions to cause algae to grow are utilized.

Aerobic lagoons accomplish purification by a combination of oxidation and synthesis in a manner similar to activated sludge. On an average they produce about 0.5 lb. of suspended solids per pound of BOD_5. Aerobic lagoons differ mainly in that time has been substituted for sludge recirculation. Effluent quality without solids separation is not high and solids separation adds cost and other significant problems.

A normal stabilization pond is not effective in removing nitrogen and phosphorus without algae removal. Even with algae removal there is little data to suggest that reliance should be placed on lagoons to control fertilization of receiving bodies of water. Removal of nutrients to low concentrations is being required in more and more regulations.

The question to be faced at this time is to what extent lagoons can consistently remove solids and will the effluents meet the requirements of the highest water uses. If not, then a challenge is issued to improve the quality of the discharge so that it is not a debit to the receiving stream. Pond effluent should not increase the total suspended solids of the receiving stream.

Bacteriological Requirement

The desirability of excluding pathogenic organisms from water is self-evident. Although modern disinfection techniques have greatly reduced the dangers from disease-causing organisms in drinking water, incomplete elimination of these agents from sewage and industrial waste poses a continuing health hazard. Even if we cannot now eliminate all of the harmful bacteria, it is reasonable to expect that good treatment along with disinfection will reduce consistently the concentration of fecal coliform organisms to 200 per 100 ml.

The popular song states that this is the dawning of the age of Aquarius. Frankly, it can better be described as one of Aquamania, or the craze for water. People who a few years ago thought water was only required for mixing drinks, now look at water as an item in which to scuba dive, swim, and on which to boat. Gazing at and walking beside water are amenities not to be overlooked. Polluted water in a lake, a river, or a reservoir is of little use for recreation or anything else. To provide satisfying outdoor recreation experiences, we will need to provide safe water. This means effluents from waste treatment processes will need to be disinfected.

The fact that in favorable locations stabilization lagoons can definitely effect high percent reduction of coliform bacteria is well-documented. Numerous articles in the literature attest to this fact. The important point of all these studies is not that the reduction of fecal bacteria was 98%, but that the effluents from many of the lagoons still contain too many fecal bacteria to meet the water quality standards unless there is several-fold dilution in the receiving waters. Many places, where land is available for lagoons, are located on streams with limited or negligible dilution capacity. The efficiency and cost of chlorinating these effluents loaded with algae is questionable. In the past, the disinfection of lagoon effluents has seldom been practiced. To effectively kill the coliform group of organisms remaining, the algae would also most likely be killed. As previously pointed out this would dischage considerable BOD-demanding matter into the stream. This renders disinfection of the effluent impracticable for many circumstances unless the algae are removed.

Some have questioned the ability of chlorination to meet adequately the future requirements for effective disinfection. It is true that the effectiveness and efficiency of chlorination is limited and it cannot be employed economically alone without good waste treatment. The better the waste treatment, the more effective is chlorination. It should be clear that in no case have other waste treatment processes been supplanted by chlorination, but chlorination supplements many other processes and has found its proper place among them. Chlorine must come into contact with a pathogen to destroy it. Well-oxidized waste treatment plant effluents free from large flocs can be freed of pathogens by chlorine, thus making the receiving water safe for recreational uses. Chlorination carries a public acceptance. The public more readily accepts the fact that wastewater has been disinfected rather than just held in a lagoon for so many days.

The phenomenon of "aftergrowth" of coliform bacteria in chlorinated wastewaters has long been recognized. This is particularly true with raw sewage or poorly treated effluents, due to the ability of *Aerobacter aerogenes* and other non-fecal coliforms to grew under such conditions. This will not be true in chlorination of the so-called "high quality" effluents that will be produced by the well-operated plants of the future.

Will the effluent from lagoons consistently meet the fecal bacteria requirements of future water quality standards? This is another challenge for lagoons.

Summary

In summary, the question is can we at the present time predetermine in the planning stage the quality of lagoon effluents as confidently as we can effluent quality from trickling filters or activated sludge plants. Do lagoons consistently produce an effluent of high quality year around and not just under the most favorable environmental conditions? Will there be a need to remove solids, nitrogen and phosphorus from the effluents before they are discharged into a stream? If the answer to the last question is "yes", treatment by lagoons might not be as economical as we would like.

Millions of people in this country who depend on surface supplies for their water, use a water supply which is partially composed of reconditioned wastewater. More and more this is true. Under these circumstances it will be necessary to require progressive improvement in both sewage treatment efficiency and in the efficiency with which surface water is processed for municipal use. Accepting the fact that there must be a progressively better job of collecting and reconditioning wastewater before its discharge, can lagoons be modified and operated to meet the future water quality standards on a sustained basis? This challenge I leave with you.

STATE OF THE ART - OXIDATION PONDS

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It is axiomatic that the goals of waste treatment are concerned with two processes, namely 1) removal of (or greatly reduce) infectious agents and 2) transform utilizable inorganic and organic substrates into stable end products. However since the degree of waste treatment required has been somewhat contentious and since increased organic loading is being experienced by many industrial and domestic systems it is imperative that a complete understanding of each waste treatment system be delineated.

This symposium on "Waste Treatment Lagoons" has presented a review of past accomplishments and has suggested areas of endeavour which should give us a better foundation for "moving ahead".

Design and Criteria

A variety of workers have generally agreed that multiple cell oxidation lagoons, operated in series, have the greatest potential for reducing the organic loading and microbial numbers in industrial and domestic wastes.

Stream standards of 25 mg/l BOD and 5 x 10^3 coliform/100 ml can be met by properly designed oxidation lagoons. Loadings of these lagoons has been established both by operating and theoretical information.

Since the efficiency of facultative lagoon systems relates primarily to the aerobic status of the upper layers of the pond, primary consideration must be given to algal growth. Several workers have suggested that visible solar energy, which is ultimately responsible for algal photosynthesis, may be the limiting parameter in oxygen production in the lagoon. Ice cover complicates this relation of solar energy to loading and the suggested relation of 1.5 langleys (1 langley = 1 gm cal per cm^2)/day/lbs of BOD/acre must be reevaluated with the operational requirement that 15 to 20 lbs BOD/acre/day be the maximum loadings for areas concerned with winter ice cover (Neel et al., 1961). Where ice cover is not a factor the values of Oswald (1963), relating solar energy to latitude and month, may be considered. He suggested that with an average light conversion efficiency of 4 per cent, 1 langley will produce sufficient algal yield to satisfy one lbs of BOD/acre/day. Others have suggested that 2 langleys are required to supply sufficient solar radiation for the satisfaction of 1 lbs BOD/acre/day (Gloyna, 1968).

Areal loadings in waste stabilization ponds, particularly in temperate climates, appears on cursory examination of the literature, to be based on experiential observations. However, assuming a location near 50° north latitude and an ice cover of four months (when loadings of 20 lbs BOD/acre/day are suggested) and the assumption that 2 langleys radiation satisfies one lbs of BOD/acre/day, then loadings up to 40 lbs BOD/acre/day would appear acceptable. A review (Fischer et al., 1968) of loadings in lagoons at or above this latitude, particularly in Canada, reveals design loadings between 10 and 45 lbs of BOD/acre/day. Where odors do occur, it is reported that either greater loadings, less langleys radiation, or longer periods of ice cover accounts for the inefficient operation. Efficiencies are generally measured in terms of BOD and coliforms reductions and thus do not reflect subtle changes in organic species being discharged.

At locations in mid-America loadings of 60 lbs/acre/day are permissible on the basis of solar radiation. The experiments of Neel et al., (1961) initially suggested this relation and their results corroborated the validity of the relationship. The use of two-celled lagoons was recommended since during ice cover and low langley production the major loadings would be to the primary cell with less treatment required of the secondary cell. Thus for example 60 lbs of BOD/acre/day could be loaded in the primary cell with less loadings being discharged to the secondary cells whereas a single-celled lagoon could be loaded at only 20 lbs/acre/day.

Recent tabulations by the FWPCA suggest that 3,700 sewage stabilization lagoons have been constructed, according to engineers' specifications, in the United States. All of these lagoons generally are square or rectangular in design and have minimum slope ratios of 1:3. A variety of techniques have been used to control erosion (Gloyna, 1968) with much of the cause of erosion being related to regulating water-depths during those times of the year when wave action causes the major damages. Most of these lagoons have the influent in the center (or slightly eccentric) to allow even distribution of the liquids and setteable solids.

Although adequate oxygen production is paramount, sludge effects in ponds must also be considered. Various investigators and operators have shown concern but little data for bottom generation of organic substrates, particularly during the summer months when fermentation by the benthic constituents causes solubilization of sludge. However the elegant experiments of Oswald et al., (1964, 1970) present definitive results on the activity in sludge. They reported that ponds designed to allow methane fermentation will dispose of about 56 lbs BOD/acre/day at 16°C without giving rise to nuisance. If temperatures are favorable and pH control is maintained up to 1,120 lbs of gas/acre/day may result at 35°C. The gas produced contains primarily methane, but hydrogen, carbon dioxide and inert elements are also present. Their experiments also suggest the importance of understanding biologic dependence on both physical (temperature) and chemical (pH) parameters in the successful operation of lagoons.

Degradation of organic molecules by microbial systems is determined by the enzymic constituents of the microbial populations and is governed by temperature, substrate availability, populational densities and other chemical characteristics of the suspending medium. Utilizable versus toxic substrates must be distinguished since they determine various populated abundances seen during the seasonal variations which occur in lagoons. Cavalier discharge of lagoons tends to disregard these considerations and promote water pollution.

Recent design criteria (Oswald et al., 1970) allows for the prevention of short-circuiting in lagoons. Proper placement of influent and effluent structures in a minimum of four cells in series prevents coliforms and organics from reaching the next cell without proper detention. Placement of structures to withdraw alternately from the center and top of ponds performed successfully for Oswald.

Seepage is a major factor to be considered in the design of sewage oxidation lagoons. Richmond (1970) suggested that seepage factors must be determined prior to the design of any lagoon in Michigan. Other workers have experienced this problem in many areas of the country and world.

Additionally in considering the physical factors in lagoon design, one must evaluate the degree of evaporation, precipitation, percolation and influent hydraulic loading (Oswald, 1963). Different geographic areas may utilize operating depths ranging from one to ten feet and detention times of a few days to twelve months. All factors must be related to the ultimate consideration of effluent quality.

An excellent review of those considerations paramount in lagoon design has been presented by Gloyna (1968). He suggested that problems in lagoon treatment are related to inadequate engineering, poor maintenance, and lack of operational supervision.

Although these problems are interrelated, the initial engineering or design must be determined by

(1) Waste characteristics.
(2) Illumination or depths of the ponds and geographic location which determines,
(3) Mean temperatures of the lagoon which regulates

or controls,

(4) The microbial species which are selected by the wastes introduced into the lagoon and thus the nutrients available for growth,

(5) Which relates to the detention times of the wastes in the lagoon, which again relates to waste characteristics.

Whether the pond functions as an aerobic, facultative or anaerobic pond is determined by the above variables. The following experiences of operators or investigators thus reflects the efficiency of these lagoons and considers these variables.

OPERATING EXPERIENCES

Domestic Wastes

The symposium on "Waste Stabilization Lagoons" held in Kansas City in 1960 reviewed the experiences of a variety of municipalities in treating sewage in lagoons. Loadings, detention times, BOD reductions and coliform die-offs were presented for many of the installations discussed. Odor, ammonia and phosphate reduction, pH changes, microbial reductions, mosquito problems, solar radiation effects, alkalinity changes, algal ecology, dissolved oxygen changes, oxygen and temperature stratification, ice cover, methane fermentation, resuspension of organic matter, hydrogen sulfide production and economics were the major parameters reported for the studies presented. An introduction to the use of stabilization lagoons for industrial wastes was also reviewed. This review will reflect only those operating experiences which differ or add to this information on stabilization lagoons.

Canadian ponds (Fisher et al., 1968), receiving organic loadings from 10 to 45 lbs BOD/acre/day gave removal efficiencies of from 65 to 90 per cent in the summer to 0 to 55 percent in the winter. Coliform reductions in excess of 99 percent were observed. Odors were common during ice break-up. Many municipalities are using an anaerobic cell of short detention time prior to discharge to a facultative cell.

Lagoons in Colorado (Svore, 1968) functioned without odor under conditions of 10 lbs BOD/acre/day with 4- to 5-foot operating depths. Increased loadings (e.g., 20 lbs BOD/acre/day) caused odor problems of a few weeks, while higher loadings (50 to 60 lbs BOD/acre/day) caused odors that persisted for as long as 3 months. Hydraulic problems were experienced with loadings of less than 20 lbs BOD/acre/day. Operational defects were related to liquid depth not being maintained, water surface inadequate and primary solids not evenly distributed.

Lagoons in the South and Southwest (Svore, 1968), loaded up to 50 lbs BOD/acre/day have little operational difficulty. However, excess blue-green algae may present odor problems, particularly if floating mats are not broken up by mechanical or natural means.

Experience with deep ponds (up to 5 m.) in Israel (Wachs and Berend, 1968) discharging into ponds of conventional depths resulted in BOD removals of 80 per cent. Loadings applied were in the area of 100 kg/hectare/day (89 lbs/acre/day). Stratification occurred during the summer with algae present to depths of 1 m. of the surface and temperature drops from 30°C at the surface to 20°C at 4 m. In the winter the gradient disappeared with mixing of bottom and top layers. Dissolved oxygen in the top layers disappeared during the winter and pH drops from 8.8 to lesser values were observed. BOD removal remained in the 80 per cent range during the winter months.

A shallow lagoon system (operating depths of 0 to 1.87 ft) was studied by Nemerow and Bryson (1963) and located at Hancock Air Force Base near Syracuse, New York. The primary cell of 1.7 acres (loaded at 130 lbs BOD/acre/day) showed 69.3 per cent removal of BOD and the secondary cell of 6.5 acres (loaded at 10.9 lbs BOD/acre/day) showed 88.3 per cent removal of BOD. This lagoon also reduced phosphates to 3 to 4 mg/liter. Much of the liquid in the cells was lost to percolation and evaporation (ca. 30 per cent of total) so that during seven of the twelve operating months no discharge was realized.

This study relates the difficulties in attempting to operate lagoons of shallow depths (apparently due to lack of compaction or sealing in the constructed cells). Growth of excess vegetation in and around the lagoon resulted with cat tails, marsh grass, small trees and weeds being evident in the cells. Mosquitoes were prevalent during the summer months and rats and muskrats caused extensive damage to the banks and dikes. Since operation of the lagoon resulted in sludge build-up, and since operating depths were shallow, it is expected that methane fermentation did not occur as one would expect in deeper ponds where anaerobiasis would be expected at the mud phase. Although the authors of this study did not indicate presence of gas formation in the lagoon, and since sludge build-up was a problem, it is assumed that methanogenic organisms were not present in large numbers.

Lagoons designed to be operated in a series were studied by Oswald et al., (1967) in California. The primary cell (1.79 acres - 8 ft depth) was loaded at 62.5 lbs BOD/acre/day; the secondary cell (1.55 acres - 8 ft depth) was loaded at 33.6 lbs BOD/acre/day; and the third or tertiary cell (1.86 acres - 4 ft depth) was loaded at 21.5 lbs BOD/acre/day. Detention times of 48, 54 and 90 days were realized respectively in the ponds. Effluent from pond 3 had a BOD from 3 to 11 mg/1, ammonia nitrogen of less than 1 mg/1 and 1.5 to 2.3 x 10^3 coliform organisms/100 ml. Euglena were the predominant algal species in cell 2 and 3 in concentrations of from 3 x 10^5/100 ml in cell 2 to 1.3 x 10^6/100 ml in the tertiary cell. Micractinium was the predominant plankton in the primary cell in concentrations of 3 x 10^6/100 ml.

Dawson and Grainge (1969) reported on lagoons in Arctic and Sub-Arctic regions. Summer operating depths of 4 to 5 ft and winter depths of 6 to 8 ft were required due to ice depths of up to 5 ft. Detention times from 8 to 12 months were used. Loadings of 20 lbs BOD/acre/day were effective in producing acceptable effluent.

Excess activated sludge was treated in an oxidation pond by Fisher and Gloyna (1965). Loadings of 56 lbs BOD/acre/day in ponds of 5-ft depths in Austin, Texas required addition of nitrogen for complete stabilization of the sludge.

Design and operation of lagoons in tropical Asia, where odors do not determine the efficiency of the system, have been studied by McGarry and Pescod (1970). Mean temperatures of 20 to 30°C allowed loadings up to 600 lbs BOD/acre/day. Detention times were vary short (1-2 days) and BOD removal was directly related to loadings. Although this system allows definitive information on loadings, where bacterial reductions and *not* algal oxygenation is the paramount feature, it does not appear applicable to temperate areas.

Industrial Waste

Porges (1963) reported that 827 stabilization ponds were being used by industry in the United States in 1962. Thirty-one industrial categories utilized these lagoons. The canning industry utilized the greatest number (29 per cent) with the meat and poultry industry ranking second (20 per cent). Proges suggested that experimental data were not sufficiently definitive to indicate effectiveness of the lagoons, however operational data suggest many successful operations.

The use of stabilization ponds in the canning industry (Canham, 1968) has been relegated primarily to holding the processing seasons waste with the ultimate addition of sodium nitrate as an electron acceptor. Recent use of spray irrigation has been followed by experiments with aerated lagoons. Few if any of these lagoons have been loaded at rates acceptable for facultative ponds and therefore data are not relevant to this area.

The use of facultative lagoons in the meat processing industry has been investigated (Steffen, 1968). Loadings from 50 lbs BOD/acre/day in South Dakota to 214 lbs BOD/acre/day in Delaware for poultry processing wastes have been investigated. Effluent from anaerobic ponds treated in facultative ponds give up to 80 percent removal in BOD.

Waste stabilization ponds have and are being used in the pulp and paper industry (Gellman and Berger, 1968). Generally the lagoon is used as a secondary treatment device. Loadings of 50 lbs/acre/day in Southern kraft mills shown 85 per cent reduction in BOD. Loadings above 50 lbs/acre/day result in reductions from 23 to 55 per cent. Multiple ponds covering 670 acres reduced a total BOD load of 44,000 lbs/day by 95 per cent. The largest basin was 470 acres and received 20,000 lbs BOD/day and discharged 2,500 lbs of BOD, removing 87 per cent or 42 lbs BOD/acre/day.

Cooper et al., (1965) investigated the treatment of organic industrial wastes in California by lagooning. Rendering plant wastes were discharged to an anaerobic lagoon with an average load of 228 lbs BOD/acre/day and detention times of about 160 days. Eighty-eight per cent reduction in BOD was observed. A second pond (facultative) was loaded at about 15 lbs BOD/acre/day with infinite detention time. Reductions in BOD were in the range of 78 per cent. Overall reduction in BOD for the two ponds was 98 per cent. The anaerobic pond had between 10^7 and 10^8 purple sulfur (*Thiopedia rosea*) organisms per ml during the period of observation. Algal counts ranged from 10^5 to near 10^7 of *Chlorella vulgaris* during the months of May through July. A variety of other heterotrophic bacteria were identified and were in the range of 10^5 total viable organisms per ml. The change from a pink to green pond was determined by both ambient temperatures and loading to the anaerobic pond.

A petroleum waste pond alternated, according to Cooper et al., (1965) from a pink color (*Chromatium vinosum present*) with loadings of 60 lbs BOD/acre/day to a green color (*Carteria present*) with loadings of about 48 lbs. BOD/acre/day. Overall reduction in BOD was 38.6 per cent removal. Phenol removal was in the range of 88.5 per cent. *Chromatium* was found to be resistant to levels of phenol up to 160 mg/liter.

Poultry manure wastes were also treated in facultative lagoons and reported by Cooper et al., (1965). Loadings of near 575 lbs of BOD/acre/day were responsible for selecting the purple sulfur organism *Thiocystis* in numbers above 10^8 per ml.

Nemerow (1967) showed 92 per cent BOD reduction in a 2-stage pond receiving poultry wastes. The first pond, loaded at 935 lbs BOD/acre/day with 7.4 days detention times gave 72.5 per cent reduction. The second pond which was 2-feet deep showed a 70 per cent reduction in filtered BOD and 90 per cent total BOD reduction. Algae were predominant in the second pond.

Gilde (1967) obtained better than 90 per cent BOD removals in ponds treating poultry processing wastes with loadings of 67 lbs BOD/acre/day.

The fourteen ponds treating domestic and textile wastes studied by Kothandaraman et al., (1968) in volume ratios of 3 to 1 and loaded at 728 lbs BOD/acre/day with detention times of 2.4 days gave BOD removals of 77 per cent.

Addition of whey wastes (Maloney et al., 1960) in lagoons normally loaded at 50 to 70 lbs BOD/acre/day in El Centro, California resulted in anaerobic conditions and essentially non-functional lagoons. It was experimentally determined by these workers that experimental ponds could tolerate combinations of domestic sewage and whey combinations of 1 to 360. Any greater concentrations of whey caused a "sick" lagoon. Additions of nitrates, although not practicable, allowed increased loadings of whey.

Water Quality Criteria

Fitzgerald and Rohlich (1958) reviewed the literature on stabilization ponds with the following summary of their effectiveness being presented. Although it may not be meaningful to present mean values in comparing a variety of experiences with the same parameters, the paucity of experimental findings on a variety of lagoons was a striking feature of the literature to 1958.

	Raw Sewage	Effluent	Reduction
	mg/1 or MPN/100 ml		per cent
BOD_5	199	21	89
Soluble P	6.4	tr	99
Organic N	26.2	4.9	81
Ammonia N	23	3.6	84
Coliform	2.5×10^7	1.5×10^5	99

The Fayette, Missouri, study (Neel et al., 1961) in which oxidation ponds were loaded at 20 to 100 lbs BOD/acre/day and discharged to a secondary cell (which was receiving in addition to the primary effluent about 15 lbs BOD/acre/day raw sewage/ resulted in the reductions presented in the following table:

The theoretical detention times in these lagoons of 2.5 to 3.0 ft depths ranged from 87 days for the cells loaded at 15 lbs BOD/acre/day to 44, 29 and 22 days for cells of 30 to 45, and 60 lbs BOD/day/acre.

These authors presented results which would seem to answer the question *viz* the relationship of stabilization lagoon effluents to water criteria. Since waste stabilization lagoons may be discharged to the water course at times

	Raw Sewage	Effluent	Reductions
	mg/1 or MPN/100		(per cent)
BOD_5	267	29.4(10.7)*	89 (96)
Organic P	12.2	2.7 (1.0)	78 (92)
PO_4	25.2	5.5 (2.3)	78 (91)
Organic N	131.7	6.6 (2.6)	95 (98)
Ammonia N	30.1	5.4 (2.1)	82 (93)
Coliform	$3.7x10^{7}$	$1.4x10^{2}$(3.7)	99.9996 (99.9999)
Algae	nil	$1.4x10^{7}$	---

other than when maximum reductions in the parameters displayed in the above table are realized, mean values may better represent estimates of the functional capabilities of these lagoons. This estimate may also be academic since stream flows and discharge volumes would more accurately reflect effects to the receiving streams.

An Australian study (Parker, 1962) showed maximum reductions obtainable through a series of eight lagoons (the first two cells were anaerobic and apparently loaded at 1,658 lbs BOD/acre/day) during summer operation. The results of this study are presented in the following table:

	Raw Sewage	Effluent	Reductions
	mg/1 or MPN/100 ml		(per cent)
BOD_5	521	10.6	98
Organic N	26.3	7.5	71
Ammonia N	32.4	18.9	42
Bacteria	$7x10^{8}$	$1.7x10^{6}$	99.8
E. coli	$6x10^{7}$	9x10	99.99
S. faecalis	$1x10^{7}$	9x10[illegible]	99.99
Algae	nil	$8.5x10^{5}$	---

Two contrasting results are of interest when one compares the Missouri study with the Australian study. Organic nitrogen is present in greater amounts in the Missouri sewage (131.7 mg/1) as compared to the Australian values (26.3 mg/1). Fitzgerald and Rohlich present mean values comparable to the Australian amounts. BOD_5 reductions were amazingly similar; however, the higher ammonia nitrogen in the Australian effluent (18.9 mg/1) as compared to the Missouri value of 5.4 mg/1 displays the difficulties encountered in making comparisons. According to a report by Golueke et al., (1967) 12 to 16 mg/1 ammonia nitrogen are the lower values necessary for algal production in high-rate ponds treating domestic wastes. Although this amount of ammonia nitrogen was apparently present in the Missouri study where the production of algal concentrations of 10^5/ml organisms (or greater) were reported, the Australian study showed final concentrations of less than 10^4 algae per ml. It thus appears that other substrate/s were limiting algal production and attempts to determine what these factor/s were seems important.

Parker (1962) reported algal counts of between 2 x 10^5 and 5 x 10^6 in a one-celled lagoons with Euglena being predominant species. In the study of a series of eight lagoons, the last cell showed Closterium as the predominant species of algae. Apparently Euglena removes ammonia nitrogen fairly well, while Closterium did not reduce appreciably the ammonia nitrogen and suggests that other substrates must be limiting its numbers.

Coliform reductions in lagoons have been studied by many workers and documented by Fitzgerald and Rohlich (1958). They indicated that many lagoons were capable of lowering the coliform organisms to less than 1 per cent of the original concentration. Neel et al., (1961) reported coliform reductions to near 10 per ml in lagoon effluents as did Parker (1962) Under proper loadings and detention times coliform reductions appear to meet most Water Quality Standards.

Geldreich et al., (1964) reported coliform reductions in a ¼ acre prison dairy farm pond. Raw sewage densities were 3.2 to 3.3 x 10^6/100 ml with reductions ranging from 85.9 to 94.4 per cent. Values ranged from 5.9 x 10^4 to 1.9 x 10^5/100 ml.

Experimental oxidation ponds (aquaria were used) were studied by Malina and Yousef (1964) who determined that 11.4 days detention reduced coliform numbers to 99.4 per cent. Coliform reductions were greater than total bacterial reductions, i.e. the rate was greater for the coliform organisms. Comparisons of total bacterial numbers with coliforms resulted in a correlation coefficient of 0.78.

Chlorination effects have been studied by Hom (1970), who determined that coliform could be selectively killed while allowing algal viability. It is equivocal whether these experimental findings would work effectively in the field.

Detention times in lagoons were studied by Meron et al., (1965) with the conclusion that longer detentions yeilded lower suspended solids,coliform and algal numbers in the effluent. Evaporation and increased salinity results with increased detention times.

Pretorius (1962) isolated coliphage from oxidation ponds. He indicated that the virus apparently does not reduce *E. coli*, but they (the virus) have a greater survival time than the bacterium.

Slanetz (1970) showed the value of ponds in series in the reduction of coliform, Salmonella and enteric viruses. The third pond in a series showed 4 to 5 log reductions in coliforms, fecal coliforms and enterococci. Most of the culture for Salmonella were negative while many of the virus cultures were positive, particularly during the colder months. The later finding is not new since investigators have previously shown that viability of viruses increases inversely to temperature.

Salmonella and Shigella organisms were studied in stabilization ponds by Cody and Tischer (1965), who indicated that Shigella were never detected in the lagoons studied. Salmonella were usually present in about one per ml numbers. Hok (1963) reported that 99.9999 per cent of *Salmonella abortis equi* were removed by 41 days at 10°C. at 20 to 30°C the equivalent is removed in 6 days.

Hsu and Kruse (1967) reported that Salmonella survive 3 to 5 days in summer and 2 to 4 weeks in winter (2°C). They found that artificially inoculated *Salmonella aborti equi* lived three but not four days in lagoons.

Wentz et al., (1967) identified *Clostridium botulinum* type F in a sewage lagoon in North Dakota.

Viraraghavan and Raman (1967) showed that oxidation pond treatment completely reduced *Mycobacterium turberculosis* populations in a pond receiving wastes from a sanitarium.

In a much needed microbial study of oxidation lagoons, Gann et al., (1968) found that 85 per cent of the bacterial populations is composed of Pseudomonas-Achromobacter-Flavobacterium group. Achromobacter was most numerous, followed by Pseudomonas and Flavobacterium.

Halvorson et al., (1968) studied oxidation of a variety of substrates by mixed lagoon populations as a function of seasonal change. Oxidative capability of these populations was greatest during the summer, but not directly related to the seasons of the year. Although according to the van't Hoff-Arrhenius relationship, there is an approximate doubling of biological activity for every 10°C rise in temperature (Lamanna and Mallette, 1959), the work of Halvorson suggests since this doubling effect was not observed that different populations with different temperature optima exist in lagoons.

McCoy (1967) reported that lagoons receiving liquid bovine manure show a change in flora from the predominantly fermentative populations. She reported that coliform and enterococci die-off rapidly in these lagoons. Stander (1963) reported that 1/3 the microbial population in lagoons in South Africa were proteolytic organisms.

The presence of secondary heterotrophs may also be expected in an oxidation lagoon. These organisms, such as protozoa, feed on primary heterotrophic bacteria (Pipes, 1961) or like the methane formers and sulfate reducers, use end products of primary heterotrophic metabolism (McCabe and Eckenfelder, 1958).

Steelman et al., (1967) suggested that concentrations of several insecticides (dursban, naled, abate and fention) under 1 mg/liter were effective in controlling Culex mosquitoes and were not bacteriocidal nor phycocidal.

Poliovirus titers from 10^5 units have been shown to be reduced to less than 10^2 units in lagoons (Christie, 1967).

Conley (1967) using *Staphylococcus aureus and Serratia marcescens* as test organisms in plastic tubes and dialysis bags, stated that these organisms do not survive beyond 48 hours in aerobic ponds, but die-off slowly in anaerobic ponds.

Cooke et al., (1960) isolated thirty non-pathogenic yeasts from sewage and noted that 20 species are associated with disease in man. Habitat of sewage was adequate for growth of these yeasts. Cooke (1963) also isolated over 100 species of filamentous fungi. Several species were potential plant pathogens. Effluent would apparently carry spores of these fungi.

Of all the organisms in oxidation lagoons, the role of algae has been most extensively studied. It is generally conceded that these autotrophs (which may also function as heterotrophs) are the primary contributors of the oxygen used by heterotrophic bacteria, in the oxidation of organic substrates.

Fitzgerald and Rohloch (1958) suggested that there is little correlation between species which become dominant in a pond and the geographic location of the pond; the degree of BOD loading; and the degree and type of sewage pretreatment. However, Neel et al., (1961) reported that phytoplankters in lagoons in Missouri were usually green algae and euglenophytes. Euglena exhibited the greatest adaptability to loading variations. Chlamydomonas and various Chlorococcales were the other abundant types noted. The Chlamydomonas and Euglena were the first types to appear in their lagoons. Algal production increased with loadings up to 60 lbs BOD/acre/day and then leveled off with greater loadings.

Studies in India (Jayangoudar and Ganapati, 1965) showed blooms of blue-green algae (primarily Microcystis) to dominate the lagoon flora. Parker (1962) indicated that Euglena appear first in heavily loaded lagoons during summer operation. Euglena, Crustacea, and Colosterium were the dominant populations in more lightly loaded lagoons.

In a study of a two-celled lagoon in Nebraska (Donoyelles, 1967) it was determined that 16 species of algae could be identified in the primary cell and 28 species in the secondary cell. *Ankistrodesmus falcatus v. acecularis* was the most prevalent species of algae. The rotifer, Brachionous, was the prime grazer of the algae.

Moawad (1968) found that filtrates of Chlorella inhibited coliform organisms while filtrates of Chlamydoomonas were stimulatory.

Kott (1968) studied the growth of algae in citrus wastes in experimental lagoons in Israel. He determined that nitrate (50 mg/1) and phosphate as PO_4 (20 mg/1) were the optimum concentrations for algal growth. Higher concentrations of these two inorganics inhibited algal growth. The source of the nitrate and phosphate was not important.

Epply and Macias (1963) in the Mohave Desert found the alga *Chlamydomonas mundana*, occurred in an anaerobic lagoon and utilized acetate produced by anaerobic bacteria. Thus the alga functioned mainly as a heterotroph, since little oxygen was produced.

A variety of organic acids, alcohols and several pesticides were studied by Gloyna (1968) to determine their toxic effects on chlorophyll production. He suggested that any calculations of algal activity in lagoons must include these toxic agents. A method for estimating lagoon algal populations by their content of chlorophyll *a* was suggested by Tschortner (1967). Sulfides (6-7 mg/1) were toxic to algae (de la O and Gloyna, 1967).

A number of investigators have identified purple sulfur bacteria in oxidation lagoons. Generally these organisms are seen in lagoons where loadings exceed design limits. A utilizable source of hydrogen sulfide, carbon dioxide and probably volatile acids account for the presence of these organisms (Holm and Vennes, 1970).

Cooper et al., (1965) identified *Thiopedia rosea* in a lagoon in California treating rendering plant waste and *Chromatium sp.* in another lagoon treating petroleum wastes. In both instances BOD lowering of a signigicant degree (88 per cent and 48 per cent) was noted and attributed to these organisms.

Parker (1962) identified Chromatium in lagoons in Australia treating domestic wastes and receiving in excess of 1000 lbs BOD/acre/day. About 80 per cent reductions in loading were noted.

May and Stahl (1967) reported the presence of Chromatium and Thiopedia in five sewage treatment ponds and two rat waste treatment ponds near Pullman, Washington.

Holm and Vennes (1970) identified three genera of purple sulfur organisms in lagoons in North Dakota receiving potato processing wastes with domestic sewage (Chromatium, Thiopedia and Thiocapsa). Thiocapsa was the dominant organism in the lagoon and reach peak populations as the levels of acetate dropped from near 1000 mg/liter to less than 100 mg/liter. Better than 90 per cent reduction in BOD was noted during the period when the purple sulfur organisms were the dominant lagoon organism.

May and Stahl (1967) and Holm and Vennes (1970) studied organic substrate utilization by three genera of pure cultures (Chromatium and Thiopedia were studied by May and Stahl; Thiocapsa and Chromatium were studied by Holm and Vennes) of purple sulfur organisms. A variety of sugars, short chain organic acids and amino acids were utilized by these organisms which may exhibit either an autotrophic or heterotrophic existance.

Ehrlich (1966) indicated that Lemna and Daphnia, which could be successfully cultivated in lagoons, were capable of removing Chlorella, *E. coli* and total bacteria.

The association of mosquitoes with oxidation lagoons has been recently explored and reviewed by Smith and Enns (1967) They indicated that previous workers found "emergent and/or overhanging vegetation as the single most important factor favoring mosquito production in oxidation lagoons". Twelve species of mosquitos were found by these authors associated with lagoons and 5 species were found to breed within the lagoon proper. They suggested that mosquito production was related to the type of wastes, dissolved oxygen content of lagoon, available food, and presence of parasites and predators. Any lagoon change which caused floating mats of algae allowed oviposition sites for mosquito production. Kimerle and Enns (1968) reported that midgets (Chironomidae) breed prolifically in lagoons with the adults becoming a nuisance in certain area (Brumbaugh et al.,). Midge populations show a direct relationship to loading of the lagoon with few if any midges found in lagoons loaded over 50 lbs BOD/acre/day in Missouri. These workers also reported on the presence of at least 60 species of aquatic insects associated with 18 central Missouri lagoons (Kimerle and Enns, 1968). As indicated previously, emergent vegetation was the single most important factor in aquatic insect production.

Culex mosquitoes were found in large numbers in lagoons in the State of Washington by Mykeebust and Harmston (1962). Twenty-one lagoons were studied and 11 were found to produce mosquitoes. Control of mosquito populations was achieved by keeping the lagoons free of emergent aquatic or flood terrestrial vegetation.

Control of *Culex pipiens* (Mallack et al., 1962) was accomplished by using simazine for weed control and malathion for mosquito control. It was found that organophosphates were more effective than chlorinated hydrocarbons for larvae control. The malantion was not detected to have an effect on lagoon bacteria or reductions in BOD.

Growth of two major genera of carp in a waste oxidation pond in India was reported by Chatterjee et al., (1967). The fish remained viable in a 3-ft pond over a two year period.

Biochemical Reactions

According to Oswald (1968) four principal biological transformations occur in oxidation ponds. These reactions are:

$$6(CH_2O)_x + 5O_2 \dashrightarrow (CH_2O)_x + 5\,CO_2 + 5H_2O + \text{energy} \quad (1)$$

$$5(CH_2O)_x \dashrightarrow (CH_2O)_x + 2CH_3COOH + \text{energy} \quad (2)$$

$$21/2CH_3COOH \dashrightarrow (CH_2O)_x + 2CH_4 + 2CO_2 + \text{energy} \quad (3)$$

$$(CH_2O)_x + CO_2 \dashrightarrow 2(CH_2O)_x + O_2 + H_2O \quad (4)$$

The designation $(CH_2O)_x$ is used to indicate carbohydrate or microbial cells. Reaction (1) occurs in the presence of oxygen and reaction (4) produces oxygen by algal photosynthesis. Reactions (2) and (3) occur under anaerobic conditions at a variety of depths in the lagoon.

In addition to the foregoing reactions which occur primarily with heterotrophic bacteria (1, 2 and 3) and slgae (4), other major transformations have been recorded in lagoons as follows:

$$(CH_2O)_x + CO_2 + 2H_2S \dashrightarrow 2(CH_2O)_x + H_2) + 2S \quad (5)$$

$$11/2CH_3COOH + SO_4 \dashrightarrow (CH_2O)_x + 2CO_2 + H_2S + 2OH- + \text{energy} \quad (6)$$

Purple sulfur organisms (May and Stahl, 1967 – Holme and Vennes, 1970) are able to utilize a variety of amino and fatty acids for growth and energy. In addition, these organisms utilize hydrogen sulfide and carbon dioxide in the photosynthetic process (Reaction 5).

Sulfate reducing bacteria (Postgate, 1963) are capable of utilizing fatty acids and sulfate with the production of carbon dioxide and hydrogen sulfide (Reaction 6). Thus two more reactions are known to occur in lagoons.

Both reactions (5 and 6) occur under anaerobic conditions with hydrogen sulfide being utilized by one group of organisms, the purple sulfur bacteria, and sulfate being reduced to hydrogen sulfide by another group of organisms, Desulfovibrio species.

On the basis of the foregoing reactions Oswald (1968) has tabulated the ". . . required conditions or levels for (the) indicated biological reaction(s) in ponds . . ." which occur according to a variety of physical and chemical factors. All reactions can apparently occur at the same time in a given oxidation lagoon. The effects of these physical and chemical factors, at any given time, determines the reaction rate at which any of his four biological transformations are occurring. Additionally, since reactions (5) and (6) may also occur in lagoons, other optima must also be considered in order to establish all aspects of lagoon microbial ecology.

The fact that all of these reactions are occurring simultaneously, but at various rates, has been shown (Holm and Vennes, 1970) in a lagoon receiving potato processing and domestic wastes.

But identifying the presence of the reactions does not delineate the predictability of each or all occurring under specific loading conditions. The sequence is somewhat predictable however in that heavily loaded ponds select purple sulfur organisms (Parker, 1962) provided a source of hydrogen sulfide is present, and algal photosynthesis seems to be inhibited by these heavy loadings. Thus photosynthetic purples, which obtain their hydrogen sulfide from either protein hydrolysis or the reactions of sulfate reducing bacteria, may select out as the predominant lagoon organisms (Holm and Vennes, 1970).

Removal of carbon dioxide from the lagoon liquid by these organisms may cause shifts in pH sufficient to inhibit growth of methane organisms. However, the mud may have sufficient buffering capacity to allow growth of these methanogenic organisms.

As shown by Parker (1962) and others, continued utilization by purple sulfur organisms of carbon dioxide and presumedly other organic substrates, eventually allows emergence of algal growth. Meanwhile production of fatty acids has allowed the growth of sulfate reducing organisms which regenerate the supply of sulfide for the growth of purple sulfur organisms. All of these organisms have pH optima for growth and thus usually one group is selected as the major lagoon population.

It seems apparent then that the removal and addition of various substrates in the lagoon governs the reaction sequence observed. Models will need to be designed to predict the outcome of this competition for substrates which are present in the lagoon. The scheme of Oswald (1968) is an excellent but incomplete approach to this whole problem of predicting ecologic changes in oxidation ponds.

Theory

Transformation of organic wastes to acceptable end products in stabilization ponds requires the combined metabolic activities of many microbial agents. The concerted efforts of these organisms has been presented in a general equation by Gloyna (1968). His experiments with laboratory ponds which relate retention time, degradation rate, temperature coefficient and temperature were presented in the following equation:

$$\frac{K_{35}}{K_T} = \frac{t_T}{t_{35}} = \theta^{(35-T)}$$

where K is reaction rate for various temperatures, t is the reaction times and T is temperature. Laboratory ponds show $\theta = 1.085$ and $K_{35} = 1.2$ for synthetic non-setteable sewage. Variations of this equation have been presented which incorporate the effects of inhibitors.

One may add to this equation the parameter of volume and express the size of a pond required (Gloyna, 1968):

$$V = fQ \frac{Y}{200} t_o\theta^{(T_o-T)}$$

where v is volume, Q is flow, Y is influent 5-day BOD, T_o is reaction time for corresponding temperature (T_o) for waste with BOD of 200 mg/1, T_o is temperature related to reaction time t_o(T_o = 35C), T is temperature for reaction time t (average temperature for coldest month) and f = algal or other toxicity factor.

Oswald (1968) has presented a simplified equation for hydraulic loading:

$L_h = Q/A$

where L_h is hydraulic load, Q is daily volume in acre inches and A is pond area in acres. His equation for organic load is:

$L_o = K(Q/A) \times BOD$

where L_o is lbs/acre/day, BOD is in mg/liter and K equal 0.226. To determine detention times the following formula is used:

$t_D = V/Q$

where V is pond volume, Q is daily flow and t_D is in days. Oswald also suggested the following formula to be helpful in considering design volumes in that:

$V = lwd$

where V is volume, l is length, w is width and d is depth, and since lw is pond surface area, AD may be substituted for V in the equation yielding the expression:

$t_D = (A/Q)d$

Marais (1964) has presented equations which allow consideration for sludge effects in BOD loading in that:

$$Y_{up} = \frac{Y_{ui}}{Kt + 1} (f_p + c_p f_s)$$

where Y_{up} is the ultimate pond BOD, Y_{ui} is the ultimate influent BOD, t is the retention time for a complete mixed system, f_p is the fraction of influent BOD to pond liquid, f_s is the fraction of influent BOD which ends in sludge layer, and c_p is the fraction of fermentation products from the sludge layer which enter the pond liquid.

This important consideration by Marais probably accounts for the inaccurate estimation of BOD reduction occurring in waste stabilization lagoons in Northern climates during winter months.

The relations of algal concentrations to pond depth has been established by Oswald (1968) to be inversely proportional to pond depth and represented by the following:

$dC_c = 2{,}000$

where d is the depth in inches of the pond and C_c is the concentration of algae in mg/1. Although theoretically a pond should not be deeper than a few feet for maximum algal growth, this increased growth results in high pH values which cause precipitation of algae and oxygen production at the bottom of the pond where methane production should be active (Oswald, 1968). From a functional point-of-view ponds should be nearer 4 feet to prevent this ecologic change.

Research Needs

Dr. Karl Wuhrmann of the Institute of Water Supply in Zurich, Switzerland at a research conference held at Rutgers University in 1964 (Heukelekian and Dondero, 1964) suggested that "... as long as we are satisfied with BOD values of river water and the oxygen balance in rivers, nothing new could be added." He suggested that satisfactory answers may be difficult to arrive at but that we are dealing with at least three independent variables:

"(1) The kinds and quantities of polluting chemicals,
(2) The types and quantities of organisms, and
(3) The rate of degradation, i.e., self-purification."

If we concur with Wuhrmann concerning the necessity for knowing something of these independent variables as they concern rivers, then it would appear we must understand these same variables as they relate to Waste Treatment Lagoons, since it is their effluents which reach the water supplies which Wuhrman has reference to.

We (Fillipi and Vennes, 1970) recently presented a paper concerning the three variables mentioned above. It was determined that Biotin, a B vitamin, is produced in rather large amounts in sewage oxidation lagoons. Specifically it was determined that up to 10,000 ng/1 of the vitamin are produced in a lagoon receiving domestic and potato processing wastes. Laboratory experimentation and isolation of a variety of bacteria from the lagoon suggested that synthesis of biotin was primarily by *Aerobacter aerogenes*, while several other organisms, a Salmonella sp. and Streptococcus sp., may have synthetic capabilities. Two photosynthetic organisms, Chorella and the purple sulfur bacterium, Thiocapsa, were the prime degraders of the vitamin.

There was no correlation between biotin concentrations and BOD. Utilization of the vitamin by Thiocapsa occurred only with Millipore-filtered lagoon samples. Contrarily, Chlorella utilized the vitamin in the presence of other lagoon microorganisms and in Millipore-filtered lagoon samples.

It was also determined that wastes in sugar beet holding ponds (from sugar manufacturing) contain up to 4400 ng/1 of biotin. Since some of these wastes, as well as wastes from potato processing and domestic lagoons, may be discharged to rivers while large quantities of the vitamin are present, the effect must be considered.

This example of lagoon ecology is, in our estimation, an example of the kinds of information needed on sewage oxidation lagoons. It must be determined what relation microorganisms and polluting chemicals have on degradation rates in oxidation lagoons and streams. Research needs must deal with these three independent variables.

It would appear useful to construct models of lagoon ecologic relations after various rates of substrate utilization have been determined. Additionally those microorganisms responsible for degradation of these substrates must be included.

More information is needed on methods of removing algae from lagoon effluents. Part of the solution may come from the use of these algae for energy sources. At any rate the continued discharge of organics (in the form of algae or other microorganisms) into streams will continue to add to the organic burden of the stream. The effects on stream biota must be determined.

Finally, in order to accomplish many of the research needs relating to waste water treatment, there must be a common ground of discussion by all disciplines concerned with the design, operation and biologic aspects of waste treatment lagoons. There is ample opportunity for contribution by all groups.

CONCLUSIONS

The present status of sewage oxidation lagoons (probably best described as facultative ponds) may be summarized by the following statements (some of which suggest areas of needed research activity).

1. Lagoon design in temperate climates relates to solar radiation in that 1 Langley/day radiation will treat 1 lb BOD/acre/day.

2. Reductions in organic loading and enteric microorganisms is best achieved by ponds in series.

3. Phosphate reductions in these ponds is not of a high order and if in fact phosphate is the limiting substrate in eutrophication of rivers and streams then more research is necessary to establish removal rates of a higher order.

4. Algal increases (minus methane production and possible other gases such as nitrogen) accounts for a major share of the transformation in organics and inorganics arriving in the lagoon. Benthic deposits make up much of the organic loading and must also be subtracted from algal yield. Techniques will need to be developed for algal removal from effluents, since this source of organics may be the limiting factor in stream eutorphication.

5. Seepage (percolation) and evaporation must be considered in lagoon design since in some cases an effluent may relate only to these two physical factors.

6. Emergent vegetation as a result of low liquid depths can be controlled by proper water levels. Mosquito production is related to the presence of this vegetation.

7. Chlorination may be effective in killing coliform organisms and although algae are more resistant to this oxidant, it is equivocal whether selective killing can be achieved in the field. Dead algae may function as immediate organic loading on the receiving stream.

8. Algae may function as heterotrophs at deeper levels in streams and under conditions of impaired solar radiation. This effect may also be seen in lagoons.

9. Mathematical simulation of liquid and benthic changes in lagoons has resulted in useful prediction however all of these formulations utilize BOD (an unreliable parameter in terms of organic species being discharged) as the main variable. More knowledge is necessary on the species of pollutants and microorganisms being discharged to streams.

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STATE OF THE ART- AERATED LAGOONS

by

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The aerated lagoons were primarily a product of the 1960s. It is true that the first aerated lagoon experiments were started in 1957; but it was not until the 1960s that aerated lagoons were recongized as valid treatment systems. The aerated lagoon at the Glatfelder Paper Co. (4) was the start. It had a 3-4 day retention period and accomplished 55% BOD reduction in the pilot plant. The full scale unit produced 60% BOD reduction with 4 days retention. Basically, the aerated lagoon was developed as an intermediate treatment system where complete treatment was not needed. Eckenfelder and O'Conner (2) were among the first to take advantage of aerated lagoons. They carried out laboratory studies using bubble aerators. It was only natural that they would use diffused aeration when they went to field studies. Unfortunately, diffused aeration was not the most efficient method for mixing and oxygen transfer in aerated lagoons. Mechanical aeration rapidly became the most widely used method for aeration in aerated lagoons. The early work at the Glatfelder Paper Co. utilized new concepts in mechanical aeration. Actually, mechanical aeration was not new; but it had not had extensive use in the United States. Activated sludge systems had used diffused aeration; but diffused aeration did not lend itself to large aeration cells. Mechanical aeration produced both mixing and oxygen transfer.

Red Bridge Subdivision

The first real aerated lagoon for the treatment of domestic sewage was constructed in the Red Bridge subdivision in Kansas City, Mo. The Red Bridge aerated lagoon parallels the development of aerated lagoons in the United States. Initially, the Red Bridge aerated lagoon was designed to treat the sewage from a shopping center and surrounding subdivision. The unit was 81 ft. in diameter at the surface and 65 ft. in diameter at the bottom with a 4 ft. water depth. The aeration time was 4 days for the sewage from 314 people. The mechanical aerator was an Infilco Vortair type mechanical aerator. This system was studied extensively by Howard Edde (3) and Bud Benjes (4). Initially, the aeration units were floating to keep a constant level of submergence with a constant oxygen transfer rate. Ice buildup during winter operations resulted in increased weight to the aerator and increased submergence. It soon became evident that a fixed aerator platform was better than a floating platform, where ice was a problem. Needless to say, once it became evident that aerated lagoons could produce a reasonable BOD reduction, efforts were directed towards producing the highest quality effluent possible. At the Red Bridge Lagoon a series of short retention period ponds were placed in operation following the aerated lagoon. No longer could aerated lagoons be considered as intermediate treatment. The effluent quality from the ponds was found to be related to the algae discharged rather than to untreated wastes or even bacteria. The algae could exert a BOD equal to the incoming wastes when there was a sudden loss in the effluent; but largely, a high quality effluent was produced.

Examination of the aeration cell indicated there was a balance between aeration and mixing. In short term aeration systems, less than 24 hours, oxygen transfer was the key factor; while in long term systems mixing was the key factor. Sawyer (5) felt that aerated lagoons should be operated as completely mixed activated sludge systems. Theory indicated that completely mixed units would have a relatively short detention period if the power requirements were to be kept to a minimum. Sawyer recognized the importance of solids production, approximately 0.5 pounds suspended solids/pound of BOD_5 removed. He felt that care must be taken if aerators in excess of 25 HP were used.

Marais and Capri (6) made a mathematical analysis of aerated lagoons and found that it should be possible to achieve 95% BOD reduction with three aerated lagoons in series with 70% less volume than a single pond. This concept should yield interesting research for the future. One of the important things to remember is that 2/3 of the total oxygen demand should occur in the first pond.

Industrial Wastes

It was in the area of industrial waste treatment that aerated lagoons have had their greatest use. Unfortunately, tha lack of sound design criteria has resulted in a conglomeration of confusing designs and systems; some of which work and some of which do not work efficiently. The pulp and paper industry has made extensive use of aerated lagoons to treat their wastes. Eckenfelder (7) examined the effect of nutrients on the stabilization paper wastes and found that BOD reduction was essentially a function of time. Increasing the length of aeration, resulted in increased BOD reduction at a decreasing rate.

The use of aerated lagoons for treating paper wastes was not enthusiastically accepted at first. Gehm (8) reported that aerated lagoons offered no advantages over activated sludge and did not produce as good an effluent quality. The Glatfelder aerated lagoon employed 6 – 60 HP surface aerators and had a 4 day aeration period. The BOD reduction ranged from 62 to 82%. In time, the Glatfelder system was converted to activated sludge to meet the load placed upon it.

The simplicity of design and operation of aerated lagoons proved to be an overriding consideration. Cost was also a key factor. DeVones, Fisher, and Morgan (9) reported on the operation of three units in the Pacific Northwest. The Weyerhaeuser Company plant at Springfield, Oregon, employed 21 acres of pond, 10 ft. deep, with an 8 day retention period. Aeration was supplied by 5 – 75 HP surface aerators and one 20 HP unit. It was estimated that these aerators added 20,000 pounds of oxygen to the wastes each day. The influent wastes had a BOD of 200 mg/l and the effluent had a BOD of 16 for 92% reduction. The Weyerhaeuser Company Plant at Cosmopolis, Washington, employs two parallel aerated lagoons of 5.5 million gallon capacity each. The aeration period is 5 days with 8 – 75 HP mechanical aerators. These wastes are very strong, averaging 4,000 mg/l BOD. The strong wastes require a clarifier following the aerator to help achieve 83 to 95% BOD reduction. The Kamloops Pulp and Paper Company plant at Kamloops, British Columbia, contained 23 acres of ponds, 10 – 12 feet deep. The 13 MGD waste flow had a 7 day aeration period. In this unit there were 4 – 60 HP and 5 – 15 HP floating aerators. The mechanical aerators experienced trouble due to ice in the winter and the low temperature also reduced BOD removal efficiency.

Haynes (10), White (11), and Laing (12) have all published papers describing the aerated lagoons at their plants. The Packaging Corporation of America's Rittman, Ohio, plant (10) employs primary sedimentation followed by four parallel aerated lagoons and four secondary sedimentation lagoons. The lagoons use 3 aerators in each cell. Two cells employ 40 HP aerators and two cells employ 30 HP aerators. With 11 days aeration the BOD drops from around 410 mg/l to 69 mg/l. An additional 7 days retention in sedimentation basins only reduces the Bod to 63 mg/l. The final effluent is passed through a short time aerated lagoon to raise the DO prior to discharge. Operationally, temperature has not posed any special problems. Accumulation of sludge required that the units be shut down, drained, and bulldozed out. There were some problems with the aerator gear boxes but nothing serious.

The Riegel Paper Company of Riegelwood, North Carolina (11) had 230 acres of ponds which were converted to aerated lagoons. Originally, 14 – 60 HP floating aerators were installed in the ponds. An additional 15 – 20 HP aerators were needed to meet increased production. The BOD reduction was 86% during tests. This system is an example of the very large installations employing massive aerators.

The textile industry has also used aerated lagoons. The Moorseville Mills plant (13) employed concrete aerated lagoons having 48 hours retention. Four 25 HP aerators were employed to transter 80 pounds of oxygen per hour each. The aeration cells were followed by sedimentation ponds having 12 hours capacity.

King and Randall (14) reported on a rather complex treatment plant for United Piece Dye Works in Bluefield, Virginia. The wastes were collected in an equalization basin prior to being treated with 800 mg/l lime and flocculated. As the flocculation proceeded, 0.4 mg/l anionic polymer was added to produce maximum flocculation. The solids were removed by sedimentation and then neutralized prior to being treated in the aerated lagoons. Four aerated lagoons were employed in series. The first cell had a two days retention with 100 HP aeration equipment. The second cell had a 20 HP floating aerator; while the third cell had two 20 HP floating aerators; and the fourth cell had a single 20 HP floating aerator. The total aeration time in all four cells was 5 days. In the summer the BOD reduction was 90 percent while in the winter it was only 50%.

Refinery wastes have been treated by aerated lagoons (15) (16) (17). The results reported were similar to those for other industries. Ling (18) found that temperature was very critical in his aerated lagoon system. The BOD removal was 90% with 4.6 days aeration at 71°F and 25% with 4.5 days aeration at 51°F. It appeared that temperature had a greater effect with soluble wastes than with insoluble wastes. Bess and Conway (19) also demonstrated the effect of temperature on aerated lagoon performance. Pilot studies showed 89 to 94% BOD reduction with 5 to 15 days aeration at 23°C. At 9°C the BOD reductions were 83–87% with 5 to 8 days aeration and 63–71% with 2.5 to 5 days aeration. Thus, they demonstrated that it was possible to trade time for temperature. As the temperature went down, a high treatment efficiency could be maintained by increasing the aeration time. The field unit was constructed with 66 hours aeration. At 35°C the BOD reduction was 54%; while it was 40% at 15°C.

Time, temperature, and oxygen are all important factors in reducing the organic load by aerated lagoons. A concentrated waste can be treated as easily as a dilute waste. Studies on a soluble and colloidal waste (20) indicated that an aeration unit having detention time of only 0.9 days can reduce the BOD from 900 mg/l to 340 mg/l. Time is not the critical criteria for removal of specific concentrations of organic matter. The specific realtionship exists between food–time–microbes. Within reasonable time periods, food concentrations, and oxygen levels, the microbial concentrations respond automatically. As long as the enviroment allows the microbes to grow, they will. The growth of microbes is directly proportional to the rate of addition of organic material. Oxygen and mixing become the dependent variables in most biological systems, expecially, aerated lagoons.

Food wastes such as milk wastes (21), potato wastes (22) (23) and pea canning wastes (24) have been treated by aerated lagoons. Olson (22) reported on the Park River, North Dakota, plant which pretreated wastes in a one acre aerated lagoon with a 25 HP mechanical aerator. The retention time varied from 6 – 14 days with 90% BOD reduction at 300 lbs BOD/acre/day loading. Dostal (23) examined three different pilot lagoons for potato wastes in the field. The results were similar to those reported eariler.

It was apparent that industry was interested in utilizing aerated lagoons. The cost of construction and the cost of operations were definitely in favor of aerated lagoons as contrasted to other treatment systems. Simplicity of operations and results produced were also important. There was no doubt that aerated lagoons were useful; but there was some confusion as to the theory of aerated lagoons. The problem was not so much in the theory as it was in the application of the theory.

Theory of Aerated Lagoons

Eckenfelder and O'Conner (2) established the basic theory for aerated lagoons through the theory of biotreatment. They found that the rate of removal of BOD was a function of the BOD remaining, $dL/dt = -KL$. This was a classical equation that had application to aerated lagoons. The oxygen uptake rate was related to the food metabolised and endogenous repiration, $dO/dt = aLr + bS$. The values of a and b are determined experimentally. Lr is the BOD removed and S is the microbial solids concentration. Since S is very small, the term bS is neglected. In long term systems bS cannot be neglected. For practical systems evaluation indicated that the effluent BOD could be predicted from the following equation:

$Le/Li = 1/(1 + Kt)$

where Le was the BOD in the effluent, mg/l.

Li was the BOD in the influent, mg/l.

K was the constant.

t was the aeration time, hours.

The value of K was found to be 0.12/day for cannery wastes.

The Red Bridge aerated lagoon experience coupled with research on biological treatment produced a single model for complete mixing activated sludge systems. McKinney (1) stated that complete mixing aerated lagoons could be expressed mathematically in the same fashion as complete mixing. Efforts were made to obtain a basic set of were merely one form of CMAS systems. The key was complete mixing. Efforts were made to obtain a basic set of equations which fitted all conditions.

Dostal (24) employed later versions of Eckenfelders equations to evaluate aerated lagoons for pea cannery wastes. The removal of BOD was related to the microbial solids, time, and effluent BOD:

$Sa - Se = K\ Se\ Xa\ t$

where Sa is the influent BOD, mg/l

Se is the soluble effluent BOD, mg/l

Xa is the mixed liquor suspended solids, mg/l

t is the time in hours.

K is a constant.

K was found to have a value of 0.018.

The value of Xa was found from the following equation:

$Xa = (So + aSr) / (1 + bt)$

where So is the initial BOD, mg/l.

Sr is the BOD removed, mg/l.

a is the metbolism constant

b is a constant

The values of a was 0.70 and b was 0.027. The effluent BOD was determined from the following equation:

$BOD_e = Se + c\ Xa$

Where BOD_e is the effluent BOD, mg/l.

Se is the soluble effluent BOD, mg/l.

c is the endogenous constant.

Xa is the MLSS, mg/l.

The value of c was 0.24.

There have been many theories developed for biological treatment; but the theories proposed by Eckenfelder and McKinney have been the most widely used. Both theories are similar and are based on application of fundamental concepts with field data. Eckenfelder's equations lean more toward field data and McKinney's equations lean toward theoretical concepts. Needless to say, both approach the same numbers from either direction. Both sets of equations produce the same results. The only serious difference lies in theory and that is not really serious. Eckenfelder indicates that the removal of food from the fluid is a function of both the microbes present and the food remaining. McKinney indicates that the removal of food is a function of the food concentration with the microbial population related to the food applied. One is direct and one is indirect. Both are there and recognized. The difference is theory. McKinney believes in the application of fundamentals and Eckenfelder believes in the fundamentals of application. These are similar; but there is a difference.

Aeration Equipment

Theory has no value unless it is translated into practice. The key to translation in aerated lagoons is the aeration equipment. The aerator must transfer the oxygen from the air to the liquid. It must mix the wastes with both the microbes and the oxygen. It must see that solids do not settle prior to discharge in the effluent. This is the function of the aerator. If it fails to meet any aspect, the entire system fails to perform as designed.

Weston and Stack (25) examined the application of mechanical aerators to the Glatfelder Paper Company plant. Using sodium sulfite, they found the mechanical surface aerator was capable of transferring 3-4 lbs O_2/HP. Eckenfelder (26) set a pattern for mechanical aeration by evaluating a unit at different oxygen levels and different temperatures. This pattern would be adopted by the manufacturers of aeration equipment because it permitted them to set the basic pattern of oxygen transfer. It has become the standard basis of design; but it is not the right way to do the job. Unfortunately, Eckenfelder's ideas have been misused by equipment manufacturers more concerned with selling aeration equipment than solving pollution problems.

Weston (27) helped develop the first modern mechanical aerator in the United States. He applied the mathematical analysis normally employed for pumps since he considered the mechanical aerator a simple turbine type pump. The oxygen transfer of turbine type aerators was found to be approximately 3.7 lbs of oxygen/HP–hr at 20°C and 0 mg/l DO. The aerators were related to the speed to the third power and the diameter to the fifth power.

The change in mechanical aeration came with the ideas of Kalinske (28). Kalinske recognized that pumpage of water past the aerator was the important parameter. Each aerator had specific transfer characteristics. Pumping of fluid past the aerator was the key to oxygen transfer with surface aerators. Unfortunately, few engineers recognized the ideas proposed by Kalinske. McWhirter (29) arrived at the same point at the same time, but did not receive recognition. He recognized that complete mixing was great in theory but was difficult to achieve. He recognized what few people were willing to accept.

A lot has been written about mechanical aeration but little recognition has been given to the pumped concept of fluid flow and oxygen transfer. This is the most important facet to oxygen transfer that has been developed in recent years. Conway and Kumbe (30) at Union Carbide developed methods for evaluating oxygen transfer in field units. Their ideas were applied to the evaluation of an aerated lagoon system in West Virginia. McKinney and Benjes (31) examined additional methods for evaluating aeration equipment. They proposed field tests as part of the specifications for aeration equipment. Unfortunately, the application of sound design concepts has been handicapped by the lack of evaluation techniques to insure that the system operated as it was designed.

Penman et al (32) have carried out the most extensive experiment on aerated lagoons to date. They examined diffused aeration as well as mechanical aeration. The data indicated that the three test systems all gave the same results. One major problem affecting all three systems was the accumulation of solids in the aerated lagoons. The long detention periods, 20 to 30 days, without adequate mixing permitted solids to settle out. Removal of the accumalted solids was considered a serious problem. Cost data indicated that diffused aeration was more economical than mechanical aeration. Winter maintenance was quite high for the mechanical aerator.

The pump concept reached a peak in the design of caged rotor aerators for treating animal manure. One of the first successful field installations was at Lawrence, Kansas in January, 1966. McKinney and Bella (33) employed a single cage rotor to aerate hog manure produced in a farrowing house. A 90 day retention period produced the desired effluent quality. The caged rotor kept the manure and microbial solids in suspension at all times while suppling the needed oxygen. It was found that oxygen transfer was related to the fluid passing through the rotor. A study at Iowa State (34) indicated the characteristics of the caged rotor aerator in an oxidation ditch. A later study at the University of Illinois (35) examined the characteristics of different types of rotor aerators.

Diffused aeration was introduced into oxidation ponds through perforated plastic tubing in the early 1960s. The plastic tubing was weighted and connected to a header system. Air was supplied over the oxidation pond from a compressor loacted on the shore. It was possible to increase tha load on oxidation ponds by the addition of diffused air. The extent of aeration per unit of pond surface was relatively low so that heavy solids were not kept in suspension.

A number of reports (37-43) have been made on treatment systems employing diffused aeration. One of the largest installations was at Regina, Saskatchewan (40) which utilized 845,000 feet of plastic tubing in two 105 acre ponds to treat 10 Imp. MGD of domestic sewage. Thirty 30 HP, 595 cfm, 9 psi air blowers were used in this installation. Approximately 0.6 cfm of air was supplies per pound of BOD applies. The system yielded 90 percent BOD reduction.

A study of two aerated lagoons near Kansas City, Missouri, (41) showed that under identical loading conditions, 10,000 PE, the effluent BOD was essentially the same, 26 mg/l and 20 mg/l, with 70 mg/l suspended solids. The two systems were physically different with one being a large oxidation pond converted to an aerated lagoon and the other being a two cell aerated lagoon with mechanical surface aeration in the first cell and diffused aeration in the second cell. Examination of the two systems indicated that the diffused aeration tubing permitted better mixing of the lagoon, allowing the algae to grow better. The tubing tended to clog with time, requiring cleaning with acid.

The success of diffused aeration prompted the application of the air gun to aerated lagoons. Essentially, the air gun is a long pipe in which air discharges at intervals to produce a shot of air creating an air lift pump up the tube. Studies on the air gun (42) have shown that they are capable of producing better than 90 percent BOD reduction. The air gun concept has produced modifications that are being used in aerated lagoons (43).

Design Concepts

To date there have been few real design concepts developed for areated lagoons. The tendency has been to merely add aeration equipment to existing oxidation ponds that have become overloaded or where land was limited. The aeration equipment has been sized on the basis of oxygen demand and transfer with little regard for adequate mixing to insure that the wastes are really mixed with the oxygen and the microbes.

The concept of a small aerated lagoon followed by a larger pond appears to be sound. The use of 24 hours aeration permits the application of the concepts of complete mixing to design the aerated lagoon. Essentially, all of the organic matter in the wastes will be converted to microbial solids. Removal of these microbial solids will produce a satisfactory effluent in the smallest possible time.

Solids are the major problem in most aerated lagoons, especially, those treating domestic sewage. It should be recognized that from 40 to 60 percent of the suspended solids in domestic sewage are non biodegradable and must be removed from the system. Metabolism of the organics in the wastes, produces more solids. The study of Penman et al (32) has shown the impact of accumulating these solids in the aeration cell. Use of inadequate mixing permitted the solids to accumulate in the aerated lagoon proper. Cleaning these solids out of the lagoons is serious problem that must be recognized in the original design.

The 24 hour aerated lagoon has both the inert solids and the microbial solids produced from metabolism. It appears that series ponds designed for solids accumulation will yield the best operation. Mechanical solids separation system could be modified to separate the solids; but they would complicate the operation of the lagoon system. Sedimentation tanks alone are not adequate as the solids concentrations are too low for rapid separation.

It should also be recognized that algae form a major portion of the solids and the BOD in ponds having long detention periods. The algae produce oxygen for metabolism but do it at the expense of producing new algae cells. In effect the growth of algae converts inorganic matter into organic solids which must be handled. It appears that with the growth of algae, the solids handling problems of long term aerated lagoons and short term aerated lagoons are the same.

Where 24 hour aerated cells are used, their design should be such that mixing is maintained throughout the entire tank. A minimum velocity of 1 fps is needed at the extreme edges of the aeration tank if solids sedimentation are to be prevented. Care should be taken to insure that the aeration equipment will provide adequate mixing with oxygen transfer. It is important that engineers employ design specifications which insure that the desired results are obtained in the field.

At the present time the lack of sufficient data prevents absolute design criteria for aerators. It suffices to say that mechanical surface aerators will transfer from 1.5 to 2.0 pounds of oxygen per horsepower aerator should be able to mix 1,000 gallons satisfactorily.

The design of diffused aeration aerated lagoons requires application of special equipment and design for each type of equipment. The equipment manufacturers must accept full responsibility for the proper application of their equipment. Performance specifications must clearly delineate responsibility for performance.

Design concepts are currently being developed for animal manure treatment systems. These systems are being studied and evaluated at the present time. Data are currently available for the design of hog manure treatment; but are not yet available for cattle manure treatment.

Conclusions

The aerated lagoon has taken its place in the array of treatment systems. It simplicity of design and operation has made the aerated lagoon a valuable addition to the waste treatment arsenal. It is not a magic treatment process. It is a modification of activated sludge and oxidation ponds to achieve a combination of these two systems. It follows fundamental concepts and is predictable in the results obtained.

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STATE OF THE ART- ANAEROBIC LAGOONS

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Anaerobic lagoons were the inevitable result of widespread use of oxidation ponds. Increased organic discharges and the need for greater economy pushed loading rates higher and higher, eventually exceeding the natural aerobic stabilization capacity of the ponds. As considered in this presentation, however, anaerobic lagoons are more than overloaded oxidation ponds. They are units intentionally designed to create anaerobic conditions to accomplish destruction of organic matter and stabilization of wastewaters. Experience has shown that anaerobic lagoons designed to create the basic conditions favorable for complete digestion will accomplish stabilization of wastewaters without nuisance odors.

Anaerobic lagoons have been used successfully to treat municipal and industrial wastewaters as well as manure slurries from livestock feeding operatings although high-temperature high-strength wastes are particularly amenable to this method of treatment. Treatment efficiencies may range from 25 to 90% although units operating satisfactorily will generally accomplish a 60-80% BOD reduction. With effluent BOD concentrations of 100-500 mg/1, depending largely upon the raw waste, effluents must receive further treatment prior to discharge to a receiving body of water. Usually the additional treatment is provided by oxidation ponds.

All lagoons designed to function anaerobically have not, however, performed in an acceptable odor-free manner. Failures have occurred in the past and will no doubt occur in the future. Inappropriate attention to basic factors including temperature, pH, the nature of the organic material, and the dispersion and mixing of the organics with properly acclimated organisms probably was the cause of many failures. Unfortunately, odors rather than poor effluent quality proclaim the failures to both expert and layman alike, and nuisance complaints are the result. Naturally, these noisome conditions have led to lack of enthusiasm by both design engineers and regulatory agency personnel for promoting the anaerobic lagoon as a usual wastewater treatment unit.

Treatment Mechanism of Anaerobic Lagoons

The anaerobic lagoon might be considered as analogous to an earthen septic tank. During the two to ten day detention, that is usually provided, solids entering the lagoon with the wastewater settle to the bottom as sludge or rise to the top as scum. This solids separation would account for a lagoon removal efficiency of 25 to 40% on a purely physical basis. Early investigators, however, recognized that anaerobic lagoons provided removals in excess of that accounted for by simple sedimentation.

Parker *et al.* (1) in 1950 reported the results of early studies with anaerobic lagoons explaining:

> *The high efficiency of the anaerobic lagoon in removing BOD is dependent on the development of digestion in the sludge layer. Organic matter is converted to methane and carbon dioxide under anaerobic conditions and the gassing of the sludge layer seeds the sewage flowing over it.*

In this manner, the organics remaining in the supernatant after sedimentation were subjected to treatment.

The underlying treatment mechanism of the anaerobic lagoon is anaerobic digestion, a process which has been subjected to intensive investigation to unravel the complex reactions and to understand the limitations involved. Digestion of complex organic material in the anaerobic environment can be simply and summarily described as occurring in two sequential stages.

In the initial stage, the complex materials such as carbohydrates, proteins, and fats are biologically converted and fragmented by processes of hydrolysis and fermentation to the simpler organic end products including aldehydes and alcohols but principally fatty organic acids. For this reason, the mixed groups of facultative and anaerobic organisms that accomplish this initial conversion, are commonly termed "acid formers." The organisms derive small amounts of energy from the reactions which in turn is converted to bacterial cell growth. Relatively little stabilization of organic matter as would be measured by BOD or COD reduction occurs in this initial stage of the conversion; the organic matter simply undergoes an antecedent breakdown in preparation for the second stage reaction.

The methane formation stage of digestion proceeds, with a favorable environment, where the first stage ends. Organic acids produced during the initial breakdown are converted by the methane-forming organisms to gaseous end products, principally methane and carbon dioxide. It is with this second conversion that the waste stabilization occurs, being directly proportional to the methane produced. The gaseous end products released in the final stage of organic digestion may escape from the liquid as in the case of hydrogen sulfide, carbon dioxide and methane, or remain in the liquid to serve as a buffering system as with ammonia and carbon dioxide.

It is generally recognized that the organisms involved in methane formation are more vulnerable to adverse environmental factors such as low temperature or low pH and particularly the presence of dissolved oxygen. The maintenance of conditions to accomplish a proper balance between the two groups of organisms is essential to prevent the development of low pH conditions by the acid formers and to accomplish relatively nuisance free treatment in the anaerobic lagoon. According to Oswald (2) design of anaerobic lagoons to obtain the conditions conducive to continuous methane formation would, in theory, be most desirable to obtain nuisance-free treatment.

Development of Design Criteria

The design criteria that exist for anaerobic lagoons are empirical in nature. The successes and failures of earlier lagoons in a particular region have served to provide the guidance for future designs.

Parker *et al.* (1) were among the first to advocate the advantages of a pond system designed specifically to operate as an anaerobic unit. Continued experience with anaerobic lagoons near Melbourne, Australia resulted in design criteria for that region. Considering normal strength sewage, it was reported (3) that anaerobic lagoons would provide about 60 to 70% BOD removal with design BOD loadings of 900-1200 lb per day/acre in summer and 675 lb per day/acre in winter. Lagoon depths of 3 to 5 feet and detention times of 2-5 days were standard at that time. Occasional extreme odor nuisances occurred at these loadings particularly at the commencement of operation of new units, although a certain amount of odor was normally present. Removal of solids accumulations were required only at 6 to 10 year intervals.

In the U.S.A. satisfactory experiences with a pilot plant in the early 1950's led to the design by Sollo (4) of an anaerobic pond to treat meat packing wastes at Moultrie, Georgia. Loading of the unit was computed on a volume basis in recognition of its similarity to a sludge digester. Average data reported for the pilot unit revealed that a BOD reduction of 65% occurred with a 4.6 day detention period and a BOD loading of 11.2 lb/1000 cu ft/day. The full scale anaerobic pond had a water depth of 14 ft and a surface area of 1.4 acres and recirculation of lagoon contents was provided. During the first four years, BOD removal efficiency averaged about 85%. Heating facilities for the lagoon were not provided and it was noted that as pond temperatures dropped below 75°F a reduction in removal efficiency could be expected.

Following these early studies by Sollo, reports indicated wide spread application of anaerobic lagooning in the meat industry (5) (6) (7) (8) (9). These anaerobic lagoons were generally designed on an organic loading basis of 15 to 20 lb BOD/1000 cu ft/day, usually with 15 ft depths and detention times of 3-6 days. In most cases these lagoons were designed to function in areas with more severe climatic conditions than those of Georgia. Pretreatment, such as grease removal, was frequently provided. Two cells arranged in parallel were often used to receive the pretreated waste and recirculation of sludge for seeding was sometimes included in the design. BOD removal efficiencies of 60 to 80% were expected and in most cases the effluents were discharged to aerobic stabilization ponds for additional treatment.

Coerver (10) reported that design cirteria for anaerobic lagoons serving small abattoirs and packinghouses in Louisiana had been developed with a somewhat different approach. Three small ponds in series were laid out to receive the wastewaters from the abattoirs. The first pond was anaerobic and received all the wastes from the slaughtering including blood and paunch manure. This pond was designed on the arbitrary basis of one acre foot for each 500 hog units of slaughtering per week. Pond depths were usually limited to 2 to 3 feet to accommodate high ground water conditions.

In the 1960's, lagoons were coming into vogue in the animal production industry. Water carried manure handling systems were being designed to serve confined feeding operations for poultry, swine, and cattle. The early manure lagoons were often mistakenly patterned after aerobic lagoons without due consideration to the characteristics of the animal waste or the principles involved in its destruction. Following field studies of operating manure lagoons, Dornbush and Andersen (11) suggested an arbitrary design limit of 5 to 10 lbs of volatile solids per day per 1000 cu ft of lagoon for South Dakota conditions. The temperature considerations were considered critical and water depths of 5-8 feet were suggested.

The results of extensive investigations of animal manure characteristics and treatment of manure slurries in anaerobic lagoons have been published by Hart and Turner (12) (13) and Loehr and others (14) (15) (16) (17). Hart and Turner (13) reported design criteria for lagooning of animal wastes in terms of cubic feet per animal recalculated from results of several investigators in different climatic locations. For example, lagoon volumes of 100-150 cu ft/hog, 10-15 cu ft/hen, and 1000-1500 cu ft/beef were reported to result in satisfactory performance for different areas of the country. However, performance of animal waste lagoons was judged largely in terms of freedom from odor because they are designed for the special purpose of destruction and stabilization of organic manures and there is seldom an overflow of liquid to create a water pollution problem.

Anaerobic lagoons have been widely adopted in regions of Canada for the treatment of municipal wastewaters. In Canada the anaerobic ponds are termed "short detention" ponds and usually have detention periods ranging from 2 to 4 days with liquid depths often maintained at 8 to 10 ft. Depths to 20 ft have been used to conserve heat where conditions are suitable. Maximum organic loadings that are permitted are 4000 lbs BOD/acre/day when the liquid depth is 10 feet (18). This may be compared with the design organic loading rate of from 30 to 50 lb/acre/day for the secondary ponds to which the short detention pond effluent is discharged.

Following extensive studies of lagoons in California, Oswald (2) has maintained that to design an anaerobic lagoon that will be odor-free, the design must result in environmental conditions favorable to methane fermentation. An important factor would include an adequate population of methane forming bacteria to break down the organic acids resulting from the initial stage of digestion. Other necessary environmental factors would include a temperature in excess of 15°C and preferably in the range of 32°C, a pH from 6.8 to 7.2, an alkalinity of about 2,000 mg/1 and the complete absence of dissolved oxygen. Greater depths and/or specific pond shapes more conducive to digestion have been suggested by Oswald. Increasing the depth of ponds to 12-14 feet will allow organic loadings up to 1,000 lbs/acre/day in the California climate.

The design criteria for anaerobic lagoons that have evolved appear to have a regional connotation and a wide variety of units have been used to express design loading rates. There is, however, one prime consideration that influences the design and acceptability of anaerobic lagoons more than any other single factor—the risk of a treatment unit that produces nuisance odors.

Design and Operation Considerations

Several considerations in the design and operation of anaerobic lagoons appear to warrant attention in order to improve removal efficiency or solve operational problems. The development of an environment which will result in adequate methane formation should apparently be the major objective in order that odor nuisances will be avoided.

Waste Characteristics - Although anaerobic lagooning has been reported to be successful with normal strength domestic wastewaters from Australia to Canada (3) (19) (20), the inherent advantages of the process would appear to be with industrial wastes where organic concentrations, both solid and dissolved, are frequently several orders of magnitude greater than those of municipal sewage.

The quality of effluent from an anaerobic lagoon is dependent to a considerable extent on the wastewater being treated. High strength wastes might be expected to yield potent effluents. BOD concentrations of 100 to 300 mg/1 occur in anaerobic lagoon effluents treating municipal wastewaters with somewhat higher concentrations resulting with industrial wastewaters. Experience has shown that effluent BOD concentrations will probably range from 40 to 70% of inflow BOD for municipal wastewaters and 10 to 40% for industrial wastewaters.

Even though meat processing wastes contain high concentrations of organic matter with BOD and suspended solids concentrations frequently in excess of 1000 mg/1, the effectiveness of anaerobic lagooning of these wastes has been well established. BOD removal efficiencies of 60-90% have been consistently demonstrated at organic loadings of 10-30 lbs BOD/1000 cu ft of lagoon. Nuisance odors have generally not been a problem.

Anaerobic lagoons have been used to a considerable cost advantage for the successful treatment of cannery wastes in Australia (21). Fruit canning wastes (apricots, peaches and pears) were mixed with both sewage treatment plant effluent and digester sludge to supply nutrients and methane forming organisms before discharge to the lagoons. In another case, lime was added to tomato and citrus wastes to maintain a pH above 7.0 before lagooning in a similar manner. The anaerobic lagoons were found to operate odor-free when BOD loadings were 600 lbs/acre/day for fruit wastes, 400 lbs/acre/day for tomato processing wastes and 200 lbs/acre/day for citrus wastes. BOD removals were 75 to 80%. Sludge accumulations have not been a problem in that a maximum depth of 9 inches was measured adjacent to the inlet after 5 years.

Digestion of solids in anaerobic lagoons accomplishes considerable stabilization of the organic matter; however, the nutrient elements of nitrogen and phosphorus are released to the lagoon liquid. For example, with a slaughterhouse waste, a 30 mg/1 organic nitrogen reduction through the anaerobic lagoon was accompanied by an increase of 30 mg/1 in ammonia (5). Thus, the effluent for anaerobic lagoons may contain high concentrations of nutrient chemicals such as nitrogen and phosphorus which are readily available to support biological growth in subsequent secondary treatment units.

Temperature - Lagoon temperature, to a large extent a climatic factor, plays an important part in the success of anaerobic lagoons. Nearly all anaerobic ponds have been observed to function better during the summer. In Australia and California adequate temperatures in excess of 15°C can be maintained with the assistance of solar radiation throughout most of the year with lagoon depths of about 3 feet. In Canada, where the major source of heat would probably be the incoming wastes, deep ponds with short detention are found to be most satisfactory.

The high temperatures of meat slaughtering wastes are probably a major factor contributing to the success of anaerobic lagoon treatment in the more severe climates. With detention periods of 3 to 5 days 80% removals were reported for a lagoon system in Canada (9). After a thick scum crust had developed on a lagoon in Minnesota effluent temperatures averaged 87°F for a weekly period even though average air temperatures were -2°F (22).

Starch refining wastes that had temperatures ranging from 91-114°F have been found particularly amenable to treatment with anaerobic lagoons. McIntosh and McGeorge (23) reported that an anaerobic cell accomplished BOD reductions from 58% in winter to 92% in summer at lagoon temperatures of 60°F and 90°F, respectively.

Covers - In northern climates where for prolonged periods the major source of heat may be the incoming wastes, floating covers have been used for anaerobic lagoons. A plastic foam raft, 3 inches thick, was installed to cover 30,000 sq ft of an anaerobic lagoon treating corn products wastes (23). The insulation value of the plastic foam was sufficient to cause snow to remain on top of the cover during the winter with liquid temperatures of 85°F underneath (24). A similar plastic foam cover has been used to prevent heat loss from a packinghouse waste lagoon(6).

With meat packing wastes a natural scum cover forms on anaerobic lagoons as a result of grease and paunch manure in the waste. This natural cover proved to be adequate insulation to prevent the lagoon temperatures from dropping below 75°F in Minnesota (5) although preliminary design had indicated that external heat would be required and a support column had been constructed in the lagoon for an underwater gas-fired heater. The heater was never installed. Subsequent studies on this anaerobic lagoon (22) determined that the grease scum layer had a thermal conductivity of 0.38 BTU/hour-°F-ft. Snow would accumulate on the surface of the scum crust.

Covers on anaerobic lagoons may be beneficial for the prevention of odors. Coerver (10) reported that anaerobic lagoons treating slaughterhouse wastes became nuisance free only after a crust had formed over the lagoon surface. It would appear that the scum covers or crusts would be beneficial in preventing adsorption of oxygen from the atmosphere thereby creating a condition more favorable for methane forming bacteria. This would appear to be particularly important for the shallow anaerobic ponds described by Coerver.

Mixing and Recirculation - Obtaining intimate contact between the raw influent and active methane forming organisms in the lagoon may be expected to result in increased BOD removals and possibly less odor. Parker and Skerry (19) have suggested that shallow lagoons bring the deposited sludge closer to the supernatent where gas evolution would achieve mixing. Dornbush and Andersen (11) stressed the importance of the mixing that occurred in animal manure lagoons with gas bubbling action and used a pump to artificially mix sludge banks deposited on the bottom of a poultry lagoon for the purpose of eliminating odors.

Recirculation of sludge solids was provided at an anaerobic lagoon treating packing house wastes in order to simulate an anaerobic contact process (25). Sludge with 2% solids was returned from the center of the bottom of the anaerobic lagoons and recycled with inflow at a ratio of 1:1 to a manhole preceeding a corner inlet near the surface of the lagoon. Lagoon temperatures ranged from 65-90°F and BOD removal efficiencies for a four-year period averaged 79.6%. A heavy sludge bank accumulated near the inlet of the lagoon.

Recirculation of supernatent has also been used to seed incoming wastes. Over 200,000 gallons of effluent were returned daily to innoculate the incoming wastewater at a corn starch plant (23). Recirculation of supernatent from a second anaerobic lagoon in series at a rate of 1.3 to 1 for the raw wastes resulted in a 5-6% increase in BOD removal efficiency in two anaerobic lagoons treating slaughterhouse wastes (22).

Inlets – Oswald (2) has suggested that special designs of the anaerobic lagoon near the inlet to create a special sludge digestion chamber have proved advantageous. A conical shaped enlargement with increased depth surrounding the inlet was one of the designs suggested. Another design would employ a submerged baffle to surround the digestion zone into which the influent was discharged. These designs had been used to prevent intrusion of oxygen into the digesting sludge when recirculation from the aerobic ponds for odor control was included in the design.

The designs suggested by Oswald to create a digestion chamber near the lagoon inlet would be expected to accomplish mixing. Enders *et al.* (7) reported that for a lagoon receiving slaughterhouse wastes a sludge blanket developed to a depth of about 5 ft; as a result the placement of the inlet near the bottom would be expected to accomplish the desired mixing. At other lagoons recirculation of sludge to the inlet accomplished the mixing (4) (25). Canham (26) reported improved results with a single raftmounted mixer placed near the inlet of a cannery waste lagoon; however, a later installation with four mixers experienced odor problems.

Solids – The extent of accumulation of solids in anaerobic lagoons will vary considerably depending primarily upon the quantity and characteristics of the wastewater, the degree of destruction of the organic material by digestion and the concentration of solids carried with the effluent.

Solids have been found to accumulate at various rates in anaerobic lagoons treating meat wastes. The shallow ponds receiving paunch manure have required solids removal after a couple of years. With preliminary treatment, the solids buildup on the lagoon bottom is reported to be extremely slow (7) (22); however, in one case a grease scum layer 1.6 ft deep reduced the lagoon volume and detention time by 17.5% (22).

The scum crusts and sludge in an anaerobic lagoon may be expected to eventually accumulate to considerable depths and require removal. In order to accomodate the use of a dragline for solids removal, Coerver (10) suggested that anaerobic lagoons be designed to be long and narrow with the length at least three times the width. When the grease and paunch cover of an anaerobic lagoon serving packing plants in Edmonton, Canada become 5 to 6 feet in thickness, removal was accomplished by burning (27). The use of pumps has also proved satisfactory for removal of sludge accumulations (4) (23). Sludge removed from the lagoons has dried without the creation of a nuisance.

Start-Up – The literature contains accounts of start-up procedures that have avoided a prolonged period of nuisance odors when an anaerobic lagoon is put into service (5) (17). These procedures approach those used for starting a sewage sludge digester including provisions for an adequate population of microorganisms (methane formers) suitable to complete the digestion and consideration for the major environmental factors of temperature and pH.

Summer would be the preferable start-up time for an anaerobic lagoon when the contents can be maintained at temperatures well in excess of the suggested 15°C minimum. Temperatures approaching 35°C would be preferable. A high initial population of methane formers must be provided with an initial seeding of sludge from a municipal sludge digester or similar source undergoing active anaerobic digestion. Lime feeding may be used to control the pH in the lagoons to the desired range of 6.8 to 7.2.

Moving Ahead – Research Needs

Experiences with anaerobic lagoons have demonstrated the lack of basic information needed to evaluate past designs. If we are to move ahead and provide the best treatment at the lowest cost, engineers will need additional information.

Temperature – Thermal considerations should receive greater attention in the design of anaerobic lagoons particularly in the colder climates. If heat sources including solar energy and heat transmitted with the incoming waste are insufficient to compensate for the heat losses that may occur, measures might include shortening the detention time of the lagoon, limiting recirculation, or tuilizing insulating lagoon covers either natural or artificial.

Limitations in the activity of the methane formers at 15°C would appear to establish the lower temperature that must be maintained throughout the lagoon, particularly in the sludge deposits on the bottom. When heat input from wastes appears insufficient to maintain this 15°C temperature, it may be possible to separate waste streams on a basis of temperature and divert cold waste streams to oxidation lagoons after sedimentation. Heat input by artificial means could also be used.

There is a need for measuring the total heat input in future studies which evaluate loading and treatment efficiencies of anaerobic lagoons. Temperature measurements in bottom sludge accumulations as well as in the overlying supernatent would determine if minimum acceptable temperatures were being maintained throughout the entire system. There appears to be a complete lack of information on the total temperature environment at present field installations.

Loading and Detention Time – The organic loading parameter is widely used as a basis for the design of anaerobic lagoons. Naturally this term may be confusing because it has been expressed in terms of both area and volume. Regardless, organic loading does not appear to be an entirely satisfactory expression because it does not consider detention time.

The computation of organic loading considers both wastewater flow and wastewater concentration. However, for a given organic loading, a higher influent concentration would be expected to result in greater removal efficiency because of the increased detention time. This has been borne out by the high removals attained by anaerobic lagoons with high strength industrial wastes even at higher organic loadings.

However, the nature of the organics either soluble or settleable that would make up the organic loading would also be of particular significance with respect to the efficiency of anaerobic lagoons. The settleable portion would be easily retained for long periods explaining substantial BOD removals in the winter when methane production would be retarded. On the other hand soluble organics would be in transit continually and the detention time of the anaerobic unit would determine the extent of stabilization.

At the present time, there is little information available concerning the nature of the BOD that remains in the effluent of anaerobic lagoons. With anaerobic contact and sludge digestion systems, the major portion of the BOD in the effluent is attributed to the solids which are carried from the systems. This, however, has not been demonstrated with anaerobic lagoons. If, for instance, a large portion of the effluent BOD is of a soluble nature, measures including recirculation or mixing as well as increasing the detention time may be required to improve the efficiency. These measures would naturally be in addition to maintaining an overall satisfactory environment for methane formation.

Sludge Accumulations and Odors – Meager information seems to point the accusing finger at sludge accumulations on the bottom of lagoons as being a major source of nuisance odors. With low temperatures or an inadequate population of methane formers, the first stage of digestion, that of acid formation, will proceed within the sludge accumulations. The resulting organic acids would quickly exceed the limited buffering within the sludge deposits and the pH would begin to drop to further limit the performance of the methane formers.

Organic acids would be expected to accumulate and an acid sludge bank or "pocket" would develop. It is hypothesized that these acid sludge banks are a major source of odors in lagoons. Odors will be produced in the acid sludge until this localized "pickling" environment is altered, either through dispersion by mixing, pH adjustment, or development of an adequate population of methane formers to breakdown the acids.

More information on the actual conditions that exist in sludge layers of lagoons that are experiencing extreme odor problems is needed. ***In situ*** measurement of sludge temperature, pH, and ORP combined with the results of analyses of sludge samples for volatile acids, alkalinity and hydrogen sulfide, would greatly assist in determining the significance of solids as the major source of odors.

In summary, while moving ahead with anaerobic lagoons research activities should be directed at more clearly defining:

(1) the thermal environment throughout lagoons,
(2) the nature of the organics in both the lagoon influent and effluent,
(3) the actual detention time of organics within the lagoon, and
(4) the relative importance of sludge accumulations as a major source of nuisance odors.

With this information, design criteria that would result in a satisfactory waste-stabilizing environment within the lagoons can be established.

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A FIELD STUDY OF THE PERFORMANCE OF WASTE STABILIZATION PONDS SERVING SMALL TOWNS

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Introduction

Kansas relies very heavily on the use of waste stabilization ponds to provide sewage treatment for most of the small rural communities. This type of treatment system is the method of choice for several reasons: 1) the initial cost of the sewage system is lower as land is almost always readily available and complex structures are not needed, 2) the skill level of the operator need not be high, and 3) with proper maintenance, the life of the treatment system is of long duration.

Kansas has long appreciated the benefits of waste stabilization ponds; however, prior to this study very little knowledge of the performance of these systems was known. Therefore, this study undertook to examine several waste stabilization ponds that have been in operation for a period of years. This paper is a presentation of the data obtained through this study.

Mechanics of the Study

Three lagoons have been studied at time of this writing. The criteria used in selection of the lagoons were: 1) the suitability of the inlet structure for flow measurement, 2) a gravity feed to the lagoon, and 3) the loading rates, both hydraulic and organic. Each lagoon was monitored for a period of approximately five days in length. A continuous record of flow was obtained by use of a "V" notch weir and a Leopold and Stevens flow recorder, model 61R, which incorporates a strip recorder and totalizer with an actuating switch for automatic sampling. The sample-actuating switch cycled at a predetermined rate of every two thousand gallons of flow. When the impulse was received from the flow recorder, the automatic sampling device first flushed the sample lines prior to taking a sample. The size of the composite sample thus obtained was adjustable by regulating the length of the sampling interval. The composite raw sewage sample was stored in an iced thermal container until taken to the laboratory.

The sample periods were not of equal length. Sample times were from 9:00 a.m. to 3:00 p.m., 3:00 p.m. to 11:00 p.m. and 11:00 p.m. to 9:00 a.m. The length of the sampling periods was selected on the basis of normal sewage flow variations with a peak being reached about noon and the minimum flow reached about 3:00 a.m. A composite sample was obtained from the raw sewage and grab samples were taken from the effluents of the primary cell and secondary cell. At the time of collection of the samples, influent, effluent, ambient temperature were recorded as well as a measurement of the dissolved oxygen in the primary and secondary cells. After collection of the samples, they were delivered to the laboratory with a time lapse of approximately one hour.

All laboratory analyses were conducted by the Environmental Health Laboratories in accordance with *Standard Methods for the Examination of Water and Wastewater, Twelfth Edition.* Analyses were conducted for ammonia, nitrate, nitrite, potassium, phosphorus, biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), pH, dissolved oxygen, total coliform, fecal streptococcus, fecal coliform, total mineral analysis and identification of the predominant phytoplankton.

Lagoon Sampling Sites

Overbrook, Kansas – Overbrook is an agriculturally oriented town with no industries capable of producing unusual loading conditions on the lagoons. The town is the site of a rural high school of 294 students. Population of the community was 719[1] as of January, 1969.

The Overbrook waste stabilization system consists of three lagoons, each approximately 1/3 of the total area. Raw sewage is diverted to the first two cells via a splitting box. The overflow from the first two cells is directed into the third cell making a series operation with primary and secondary treatment. The lagoons are operated at the five foot depth.

Overbrook has a chronic problem of greatly increased flows during and after periods of precipitation. Rains totaling 1½ inches occurred during the course of the study and the sewage flow increased to such a degree that the flow exceeded the capacity of the flow measuring device. Ground water was thought to be scanty in this area, but unusually high flow rates are encountered at all times, apparently because of the continuous infiltration. The sewage appears to be diluted as evidenced by the low five day BOD test results and calculated flow per person. The source of the extra water could not be isolated but roof drains and foundation drains have been reported to be utilized and several sewer line crossings of drainage courses exist.

Eskridge, Kansas – Eskridge is similar to Overbrook in many aspects. Eskridge is an agriculturally oriented town with no industries. The town is the site of a rural high school of 266 students. The population of the community was 540[2] as of January, 1969.

The Eskridge sewage disposal system has three cells with the two primary cells having approximately 50% of the total 5.98 surface acres. The first two cells are operated in parallel with raw sewage going to both cells. The last cell is operated in series with the first two, thus giving primary and secondary treatment.

The sewage flows from Eskridge were not adequate to produce an effluent from the final cell. The last cell was approximately one foot below the lowest effluent weir. The primary cells were operated at the five foot depth.

Shawnee County Sewer District No. 2–The sewer district is located on the outskirts of Topeka and serves the area adjacent to Forbes Air Force Base. This district contains within its boundaries the city of Pauline, population 100, many trailer courts and several small industries. An accurate count of the population served was impossible due to the diverse nature and rambling structure of the sewer district, but it is believed to be about 2100 people.

The sewer district's waste stabilization system consists of three cells. Each cell makes up about 1/3 of the total 7.19 acres. The first two cells are operated in parallel with the third cell in series. All cells were operated at the five foot depth.

The waste stabilization ponds were heavily overloaded. The ponds receive wastes from about twice as many people as they were originally designed for. The overloading is increased by the fact that infiltration is a major problem in wet periods.

Lagoon Study Overbrook, Kansas

Design Population = 950

Present Population = 719

Design:

Primary Cells

Cell A

Surface Area = 81.021 sq. ft.
= 1.86 acres

Volume @ 5' depth = 351,650 cu. ft.
= 2,630,342 gal.

Cell B

Surface Area = 87,912 sq. ft.
= 2.02 acres

Volume @ 5' depth = 382,773 cu. ft.
= 2,863,142 gal.

Secondary Cell

Surface area = 93,636 sq. ft.
= 2.15 acres

Volume @ 5' depth = 409,482 cu. ft.
= 3,410,985 gal.

Total Surface Area = 6.03 acres

Total Volume = 8,904,469 gal.

Surface Area Distribution

64.3% primary treatment
35.7% secondary treatment

Average Dry Daily Input of Raw Sewage = 119,640 gal.
Average Detention Time = 74.4 days (no losses)
Average Daily 5-Day BOD = 174.16 lbs.
= 28.9 lbs. BOD$_5$/acre pond surface
= 47.97 lbs. /primary loading

Average daily output BOD$_5$ per person = .24 lbs. BOD$_5$ daily

Average daily sewage flow per person = 166.3 gal.

Table 1. Chemical data raw sewage, Overbrook

Date	Time	Type of Sample	5 day BOD mg/1	COD mg/1	NH_3(N) mg/1	NO_3(N) mg/1	NO_2(N) mg/1	PO_4 mg/1	K mg/1	pH
4-15	11 am - 4pm	Comp.	245	400	13.0	0.15	.008	2.40	12.0	7.1
4-16	9 am	Grab	150	260	15.0	0.10	.010	0.41	10.0	7.5
4-16	9 am - 3pm	Comp.	210	350	11.0	0.20	.009	2.30	11.0	7.1
4-16	3 pm - 11pm	Comp.	140	300	09.2	1.40	.510	0.44	10.0	7.7
4-16/17	11 pm - 9am	Comp.	075	140	10.0	3.30	.252	0.16	08.0	7.6
4-17	9 am -2:30pm	Comp.	225	360	11.0	0.05	.009	0.43	11.0	7.6
4-20	9 am - 3pm	Comp.	170	416	07.4	3.00	.350	0.58	08.6	7.2
4-20	3 pm - 11pm	Comp.	110	238	08.0	5.40	.264	0.26	06.5	7.6
4-20/21	11 pm - 9am	Comp.	050	080	07.1	6.50	.188	0.14	05.2	7.6
4-21	9 am - 3pm	Comp.	115	260	06.4	3.60	.410	0.42	08.0	7.2
4-21	11 pm	Grab	110	220	06.0	0.15	1.200	0.46	08.0	7.6
4-21/22	11 pm - 9am	Comp.	032	120	06.7	5.60	.360	0.15	05.0	7.5
4-22	9 am - 3pm	Comp.	100	180	06.0	0.35	1.000	0.40	08.2	7.8
AVERAGE			155.4	255.7	09.14	2.29	.352	0.677	08.5	7.47

Table 2. Chemical data primary cells, O[illegible]brook

Date	Time	Type of Sample	5 day BOD mg/1	COD mg/1	NH_3(N) mg/1	NO_3(N) mg/1	NO_2(N) mg/1	PO_4 mg/1	K mg/1	pH	D.O.
4-20	3 pm	Grab	10.	50.	4.1	1.3	.206	.26	8.6	7.9	--
4-20	11 pm	Grab	14.	20.	4.2	1.5	.222	.21	8.4	7.8	2.8
4-21	9 am	Grab	18.	69.	4.5	1.5	.220	.24	8.0	7.7	1.9
4-21	3 pm	Grab	12.	60.	4.1	1.3	.196	.30	8.2	8.0	2.6
4-21	11 pm	Grab	15.	70.	4.6	1.3	.222	.26	8.2	7.7	2.5
4-22	9 am	Grab	11.	60.	4.4	1.3	.204	.26	8.0	7.9	3.9
4-22	3 pm	Grab	13.	50.	4.4	1.4	.212	.42	8.4	8.0	6.0
AVERAGE			13.3	54.1	4.33	1.37	.212	.279	8.26	7.86	3.28

Table 3. Chemical data effluent final cell, Overbrook

Date	Time	Type of Sample	5 day BOD mg/1	COD mg/1	NH$_3$(N) mg/1	NO$_3$(N) mg/1	NO$_2$(N) mg/1	PO$_4$ mg/1	K mg/1	pH	D.O.
4-15	4 pm	Grab	19.	90.	1.8	.45	.176	.18	10.	8.9	--
4-16	9 am	Grab	23.	80.	2.2	.40	.170	5.9	11.	8.9	--
4-16	3 pm	Grab	16.	90.	1.9	.45	.172	.20	10.	9.0	10.8
4-16	11 pm	Grab	24.	70.	2.2	.45	.164	.19	10.	8.9	8.3
4-17	9 am	Grab	16.	110.	1.9	.45	.164	.15	10.	9.0	9.6
4-17	3:30 pm	Grab	22.	80.	1.8	.38	.162	5.8	10.	9.1	11.9
4-20	3 pm	Grab	17.	89.	1.7	.50	.164	.22	9.5	8.9	14.0
4-20	11 pm	Grab	17.	79.	2.1	7.0	.180	.22	9.5	8.8	12.3
4-21	9 am	Grab	22.	90.	2.5	.55	.184	.26	9.3	8.9	11.9
4-21	3 pm	Grab	21.	120.	1.4	.60	.176	.22	9.5	9.1	17.1
4-21	11 pm	Grab	13.	70.	1.9	.55	.188	.18	9.1	9.1	14.8
4-22	9 am	Grab	17.	90.	1.5	.70	.182	.18	8.9	9.0	--
4-22	3 pm	Grab	32.	120.	1.6	.79	.196	.20	9.1	9.2	13.0
AVERAGE			19.92	90.62	1.88	1.02	.175	1.07	9.68	8.98	12.37

Table 4. Bacterial data, Overbrook

Influent Raw Sewage

Date	Time	Sample Number	TC/100 ml.	FC/100 ml.	FS/100 ml.
4-15	11:00am-4:00pm	1	170 X 10^6	6.4 X 10^6	300 X 10^3
4-16	9:00 am	2	24 X 10^6	16 X 10^6	2.7 X 10^6
4-16	9:00am-3:00pm	3	300 X 10^6	56 X 10^6	200 X 10^3
4-16	3:00pm-11:00pm	4	44 X 10^6	4.3 X 10^6	LT 100X10^3
4-16/17	11:00pm-9:00am	5	20 X 10^6	4.8 X 10^6	900 X 10^3
4-17	9:00am-3:30pm	6	11 X 10^6	4.1 X 10^6	300 X 10^3
4-20	9:00am-3:00pm	7	20 X 10^6	3.0 X 10^6	700 X 10^3
4-20	3:00pm-11:00pm	8	4.9 X 10^6	1.3 X 10^6	300 X 10^3
4-20/21	11:00pm-9:00am	9	4.1 X 10^6	1.4 X 10^6	200 X 10^3
4-21	9:00am-3:00pm	10	42 X 10^6	7.4 X 10^6	200 X 10^3
4-21	11:00 pm	11	4.9 X 10^6	.60 X 10^6	100 X 10^3
4-21/22	11:00pm-9:00am	12	21 X 10^6	6.6 X 10^6	200 X 10^3
4-22	9:00am-3:00pm	13	15 X 10^6	1.4 X 10^6	100 X 10^3
	AVERAGE		52.38X10^6	8.72 X 10^6	484.6 X 10^3

Primary Cells

Sample Number	TC/100 ml.	FC/100 ml.	FS/100 ml.
7	120 X 10^3	23 X 10^3	5. X 10^3
8	900 X 10^3	57 X 10^3	18 X 10^3
9	150 X 10^3	51 X 10^3	8 X 10^3
10	63 X 10^3	40 X 10^3	1 X 10^3
11	25 X 10^3	10 X 10^3	2 X 10^3
12	110 X 10^3	51 X 10^3	12 X 10^3
13	140 X 10^3	45 X 10^3	5 X 10^3
	215 X 10^3	39.6 X 10^3	7.3X10^3

Table 5. Bacterial data effluent final cell, Overbrook

Date	Time	Sample Number	TC/100 ml.	FC/100 ml.	FS/100 ml.
4-15	4:00 pm	1	35. X 10^3	LT 1000	LT 1000
4-16	9:00 am	2	1. X 10^3	LT 1000	LT 1000
4-16	3:00 pm	3	2. X 10^3	LT 1000	LT 1000
4-16	11:00 pm	4	1. X 10^3	LT 1000	LT 1000
4-17	9:00 am	5	2. X 10^3	LT 1000	LT 1000
4-17	3:30 pm	6	.30 X 10^3	200	.10 X 10^3
4-20	3:00 pm	7	5.8 X 10^3	1100	.13 X 10^3
4-20	11:00 pm	8	9.6 X 10^3	2700	.51 X 10^3
4-21	9:00 am	9	10 X 10^3	3400	.36 X 10^3
4-21	3:00 pm	10	2.8 X 10^3	700	.20 X 10^3
4-21	11:00 pm	11	3.9 X 10^3	400	1.2 X 10^3
4-22	9:00 am	12	5.4 X 10^3	1100	.30 X 10^3
4-22	3:00 pm	13	3. X 10^3	200	170 X 10^3
	AVERAGE		6.29 X 10^3	1225	2.16 X 10^3

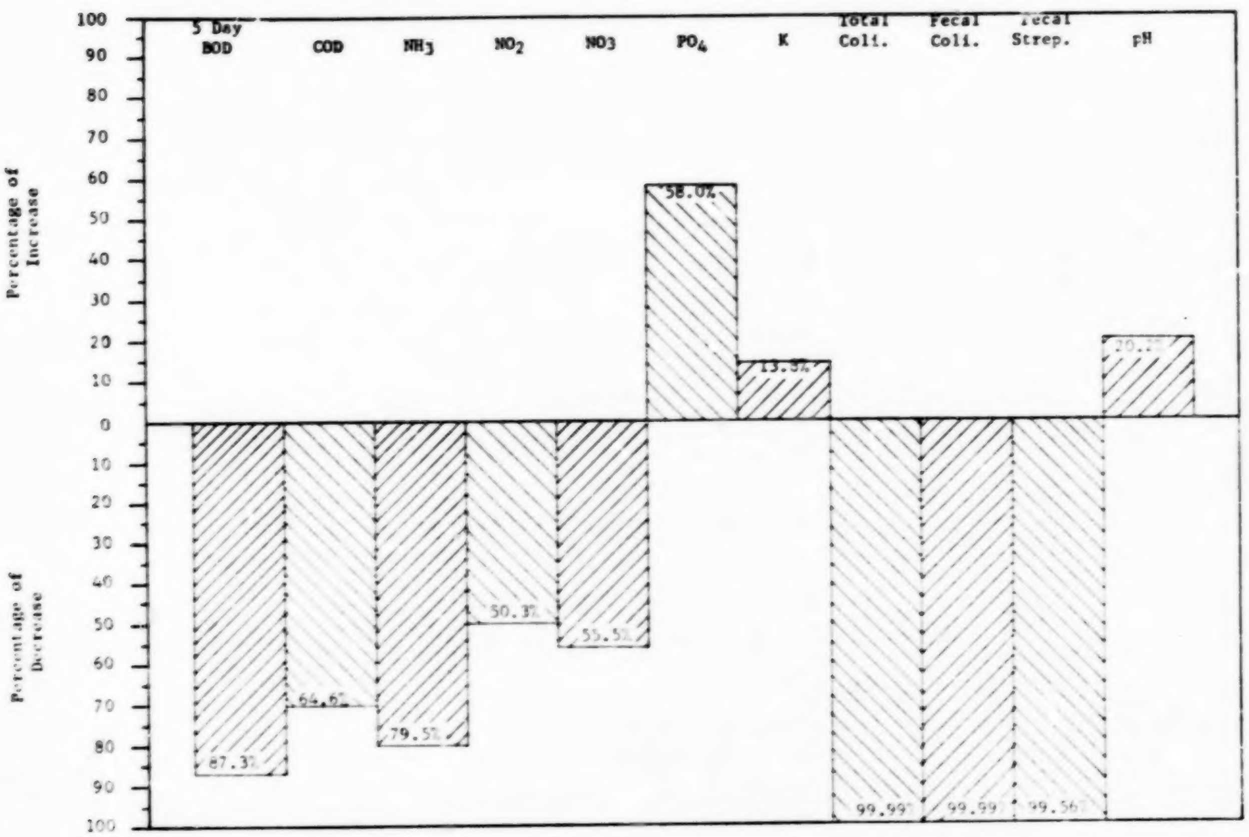
Table 6. Data summation Overbrook, Kansas

	BOD_5 mg/1	COD mg/1	NH_3 as (N) mg/1	NO_2 (N) mg/1	NO_3 (N) mg/1	PO_4 mg/1	K mg/1	Total Coliform Per 100 ml.	Fecal Coliform Per 100 ml.	Fecal Strep. Per 100 ml.	pH	D.O. mg/1
Averages for Influent	155.4	251.7	9.14	.352	2.29	.677	8.50	52.38 X 10^6	8.72 X 10^6	484.6 X 10^3	7.47	---
Averages for Primary Pond	13.3	54.1	4.33	.212	1.37	.279	8.26	215.0 X 10^3	39.6 X 10^3	7.3 X 10^3	7.86	3.28
Averages for Effluent	19.92	90.62	1.88	.175	1.02	1.07	9.68	6.29 X 10^3	1.225X10^3	2.16 X 10^3	8.98	12.37

	BOD_5	COD	NH_3 as N	NO_2 (N)	NO_3 (N)	PO_4	K	Total Coliform	Fecal Coliform	Fecal Strep.	pH	D.O.
Influent to Primary	91.5% R*	78.9% R	52.7% R	39.8% R	40.7% R	58.8% R	2.9% R	99.6% R	99.6% R	98.5% R	5.2% I*	No Data
Influent to Effluent	87.3% R	64.6% R	79.5% R	50.3% R	55.5% R	58.0% I	13.8% I	99.99% R	99.99% R	99.56% R	20.2% I	No

*R - Represents percentage reduction
*I - Represents percentage increase

Percentage of increase or decrease raw influent/final effluent, Overbrook



Lagoon study Eskridge, Kansas

Design Population = 950

Present Population = 593 P. E.

Design:

Primary Cells

Cell A

Surface area = 66,976 sq. ft.
= 1.54 acres

Volume @ 5' depth = 300,992 cu. ft.
= 2,507,263 gal.

Cell B

Surface area = 65,140 sq. ft.
= 1.495 acres

Volume @ 5' depth = 289,459 cu. ft.
= 2,411,193 gal.

Secondary Cell

Surface area = 127,950 sq. ft.
= 2.95 acres

Volume @ 5' depth = 620,167 cu. ft.
= 4,886,919 gal.

Total Surface Area = 5.98 acres

Total Volume = 9,805,375 gal.

Surface Area Distribution

50.6 primary treatment
49.4 secondary treatment

Average Daily Input of raw sewage = 75,520 gal/day
Average Detention Time = 129.8 days (no losses)
Average Daily 5-Day B.O.D. = 132.5 lbs.
= 22.15 lbs/total surface area
= 44.16 lbs/primary loading

Average daily output BOD_5 per person = .22 lb. BOD_5 pc/pd

Table 7. Chemical data raw sewage, Eskridge

Date	Time	Type of Sample	5 Day BOD mg/1	COD mg/1	NH$_3$(N) mg/1	NO$_3$(N) mg/1	NO$_2$(N) mg/1	PO$_4$ mg/1	K mg/1	pH
5-5	3:30pm-9:30pm	Comp.	240	488	22	.1	.018	8.0	19.0	7.6
5-6	9:30pm-7:00am	Comp.	135	345	22	.15	.008	1.6	21.0	7.7
5-6	3:00pm	Comp.	260	520	22	.25	.01	54	32.0	7.6
5-6	3:00pm-10:00p	Comp.	185	345	23	.20	.138	3.6	22.0	7.7
5-7	10:00pm-8:00a	Comp.	120	290	29	.55	.340	2.8	21.0	7.9
5-7	8:00am-3:00pm	Comp.	205	540	34	.15	.03	11	20.0	8.0
5-7	3:00pm-11:00p	Comp.	280	530	26	.10	.02	38	18.0	7.8
5-8	11:00pm-10:30	Comp.	115	300	23	.15	.028	21	18.0	8.1
5-8	10:30am-4:00p	Comp.	230	480	23	.10	.02	48	15.0	7.4
5-11	10:30am-4:00p	Comp.	225	500	18	.10	.02	52	17.0	7.8
5-11	4:00pm-10:00p	Comp.	320	750	11	.10	.176	54	15.0	8.0
	AVERAGE		210.45	462	23	.17	.073	26.7	19.8	7.8

Table 8. Chemical data primary cells, Eskridge

Date	Time	Type of Sample	5 Day BOD mg/1	COD mg/1	NH$_3$(N) mg/1	NO$_3$(N) mg/1	NO$_2$(N) mg/1	PO$_4$ mg/1	K mg/1	pH	D.O.
5-5	9:30 pm	Grab	41	213	1.0	.6	.56	.11	18.0	9.1	15.6
5-6	7:00 am	Grab	32	152	1.3	.5	.530	.12	18.0	9.0	6.8
5-6	3:00 pm	Grab	51	170	2.0	.40	.430	.24	18.0	9.1	12.6
5-6	10:00 pm	Grab	45	180	2.0	.40	.430	.24	18.0	9.0	7.8
5-7	8:00 am	Grab	46	200	2.2	.35	.390	.20	18.0	8.8	1.7
5-7	3:00 pm	Grab	55	180	1.2	.25	.410	.34	18.0	9.0	7.5
5-7	11:00 pm	Grab	45	170	4.4	.20	.38	8.0	18.0	8.6	2.0
5-8	10:30 am	Grab	52	190	4.1	.20	.390	7.7	15.0	8.8	5.8
5-8	4:00 pm	Grab	38	176	4.2	.10	.390	11.	18.0	8.5	13.7
5-11	4:00 pm	Grab	38	190	2.3	.15	.39	5.2	17.0	8.8	15.3
5-11	10:00 pm	Grab	27	180	2.1	.15	.32	4.8	18.0	8.8	10.4
AVERAGE			42.7	181.9	2.61	.30	.42	3.45	17.6	8.8	9.0

Table 9 Chemical data effluent final cell, Eskridge

Date	Time	Type of Sample	5 Day BOD mg/1	COD mg/1	$NH_3(N)$ mg/1	$NO_3(N)$ mg/1	$NO_2(N)$ mg/1	PO_4 mg/1	K mg/1	pH	D.O.
5-5	9:30 am	Grab	23	152	.13	.20	.018	.08	15.0	9.9	12.2
5-6	7:00 am	Grab	26	142	.18	.25	.014	.04	15.0	9.8	8.5
5-6	3:00 pm	Grab	33	145	.22	.25	.012	.09	15.0	9.9	12.7
5-6	10:00 pm	Grab	31	160	.22	.20	.02	1.5	15.0	9.8	7.6
5-7	8:00 am	Grab	31	180	.30	.15	.02	.08	15.0	9.5	5.3
5-7	3:00 pm	Grab	45	210	.75	.25	.016	.10	16.0	9.9	8.5
5-7	11:00 pm	Grab	44	220	.19	.15	.018	2.1	16.0	9.5	2.9
5-8	10:30 am	Grab	42	190	.19	.10	.02	2.0	15.0	9.7	10.7
5-8	4:00 pm	Grab	39	254	.16	.10	.02	1.9	16.0	9.2	8.0
5-11	4:00 pm	Grab	29	160	.19	.10	.02	1.3	15.0	9.7	11.6
5-11	10:00 pm	Grab	35	200	.19	.10	.02	1.8	15.0	9.6	4.3
AVERAGE			34	183	.24	.16	.018	.99	15.2	9.7	8.3

Table 10. Bacterial data, Eskridge

Influent Raw Sewage

Date	Time	Sample Number	TC/100 ml.	FC/100 ml.	FS/100 ml.
5-5	3:30pm-9:30pm	1	370X10^{6}	43 X 10^{6}	20 X 10^{3}
5-6	9:30pm-7:00am	2	530 X 10^{6}	33 X 10^{6}	1.5 X 10^{6}
5-6	3:00 pm	3	50 X 10^{6}	4.2 X 10^{6}	1 X 10^{4}
5-6	3:00pm-10:00pm	4	82 X 10^{6}	7.5 X 10^{6}	20 X 10^{4}
5-7	10:00pm-8:00am	5	45 X 10^{6}	10 X 10^{6}	40 X 10^{4}
5-7	8:00am-3:00pm	6	64 X 10^{6}	35 X 10^{6}	10 X 10^{4}
5-7	3:00pm-11:00pm	7	34 X 10^{6}	7 X 10^{6}	64 X 10^{4}
5-8	11:00pm-10:30a	8	18 X 10^{6}	8 X 10^{6}	78 X 10^{4}
5-8	10:30am-4:00pm	9	49 X 10^{6}	9.2 X 10^{6}	30 X 10^{4}
5-11	10:30am-4:00pm	10	99 X 10^{6}	18 X 10^{6}	10 X 10^{4}
5-11	4:00pm-10:00pm	11	330 X 10^{6}	18 X 10^{6}	16 X 10^{4}
	AVERAGE		151 X 10^{6}	17.5 X 10^{6}	26.4 X 10^{4}

Primary Cells

Sample Number	TC/100 ml.	FC/100 ml.	FS/100 ml.
1	15 X 10^{3}	3 X 10^{3}	7.1 X 10^{3}
2	40 X 10^{3}	9 X 10^{3}	1 X 10^{3}
3	28 X 10^{3}	2 X 10^{3}	1 X 10^{3}
4	120 X 10^{3}	34 X 10^{3}	4 X 10^{3}
5	220 X 10^{3}	70 X 10^{3}	4 X 10^{3}
6	60 X 10^{3}	30 X 10^{3}	2.2 X 10^{3}
7	45 X 10^{3}	41 X 10^{3}	2.3 X 10^{3}
8	40 X 10^{3}	18 X 10^{3}	2.7 X 10^{3}
9	54 X 10^{3}	4.7 X 10^{3}	1.2 X 10^{3}
10	62 X 10^{3}	6.1 X 10^{3}	.21 X 10^{3}
11	37 X 10^{3}	1.7 X 10^{3}	.2 X 10^{3}
	65 X 10^{3}	19.95X10^{3}	8.16X10^{3}

Table 11. Bacterial data effluent final cell, Eskridge

Date	Time	Sample Number	TC/100 ml.	FC/100 ml.	FS/100 ml.
5-5	9:30 am	1	LT 100	LT 100	LT 100
5-6	7:00 am	2	LT 100	LT 100	LT 100
5-6	3:00 pm	3	LT 10	LT 10	30
5-6	10:00 pm	4	LT 10	LT 10	20
5-7	8:00 am	5	200	10	40
5-7	3:00 pm	6	200	LT 100	200
5-7	11:00 pm	7	500	500	250
5-8	10:30 am	8	200	200	100
5-8	4:00 pm	9	200	100	100
5-11	4:00 pm	10	50	LT 10	40
5-11	10:00 pm	11	LT 10	LT 10	LT 10
	AVERAGE		143	104	67.2

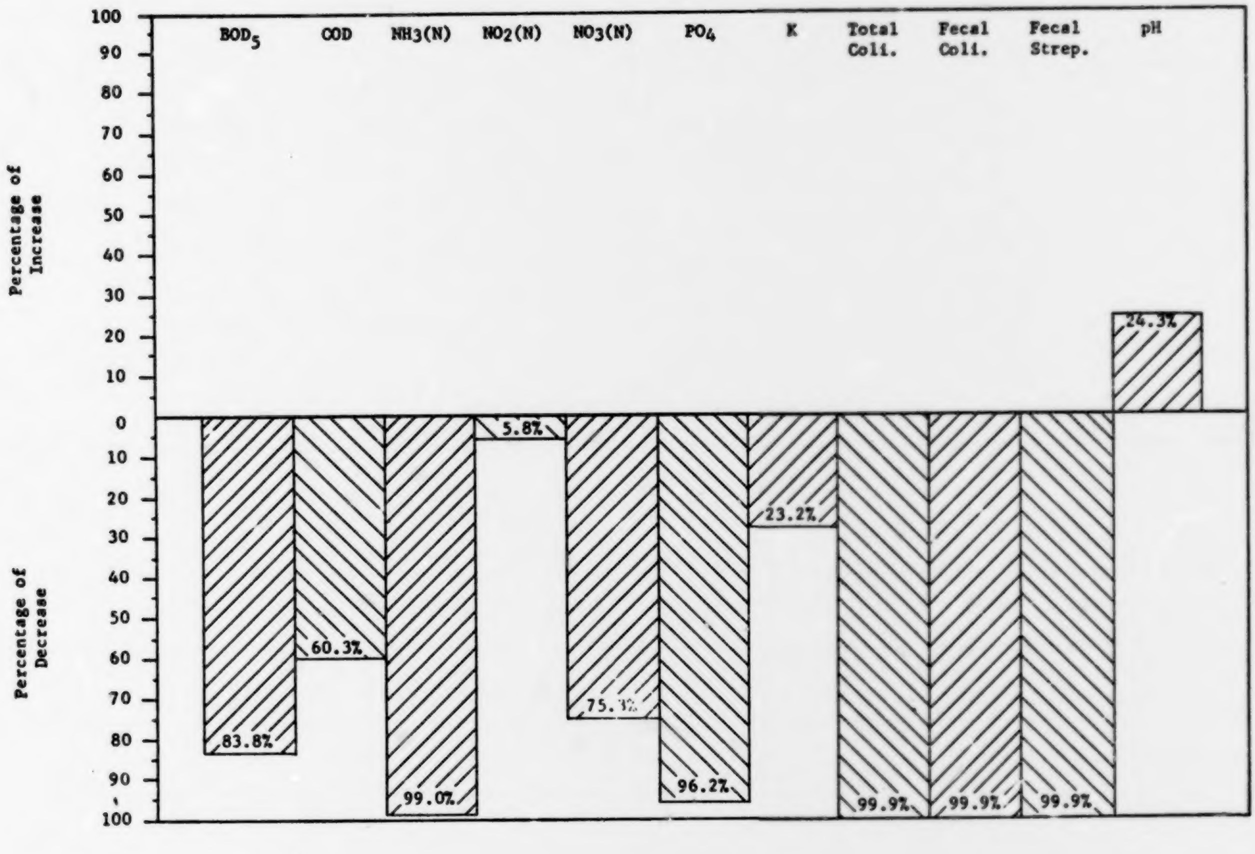
Table 12. Data summation, Eskridge, Kansas

	BOD_5 mg/1	COD mg/1	NH_3 as N mg/1	NO_2 (N) mg/1	NO_3 (N) mg/1	PO_4 mg/1	K mg/1	Total Coliform Per 100 ml.	Fecal Coliform Per 100 ml.	Fecal Strep. Per 100 ml.	pH	D.O. mg/1
Averages for Influent	210.5	462.0	23.0	.17	.073	26.7	19.8	151×10^6	17.5×10^6	26.4×10^4	7.8	---
Averages for Primary Pond	42.7	181.9	2.61	.30	.42	3.45	17.6	65×10^3	19.9×10^3	8.16×10^3	8.8	9.0
Averages for Effluent	34.0	183.0	0.24	.16	.018	0.99	15.2	143	104	294	9.7	8.3

	BOD_5	COD	NH_3 as N	NO_2 (N)	NO_3 (N)	PO_4	K	Total Coliform	Fecal Coliform	Fecal Strep.	pH	D.O.
Influent to Primary	79.7% R*	60.6% R	88.6% R	176.4% I*	575.0% I	83.3% R	11.7% R	99.9% R	99.8% R	96.9% R	12.8% I	---
Influent to Effluent	83.8% R	60.3% R	99.0% R	5.8% R	75.3% R	96.2% R	23.2% R	99.9% R	99.9% R	99.9% R	24.3% I	---

*R - Represents percentage reduction
*I - Represents percentage increase

Percentage of increase or decrease raw influent/final effluent, Eskridge



Lagoon study Shawnee Co. S.D. 2

Design Population = 1150

Present Population = 2100

Design:

Primary Cells

Cell A

Surface area = 103,100 sq. ft.
= 2.36 acres

Volume @ 5' depth = 456,000 cu. ft.
= 3,415,000 gal.

Cell B

Surface area = 106,400 sq. ft.
= 2.42 acres

Volume @ 5' depth = 475,000 cu. ft.
= 3,555,000 gal.

Secondary Cell

Surface area = 106,200 sq.ft.
= 2.95 acres

Volume @ 5' depth = 479,000 cu. ft.
= 3,580,000 gal.

Total Surface Area = 7.19 acres

Total Volume = 10,550,000 gal.

Surface Area Distribution:

66.4% primary treatment
33.6% secondary treatment

Average Daily Input of raw sewage = 292,320 gal./day
Average Detention Time = 36 days
Average Daily 5-Day BOD = 273.4 lbs.
= 38.02 lbs./total surface area
= 57.4 lbs/primary loading

Average Daily Output BOD_5 per person=.13 lbs. BOD_5 pc/pd

Average Daily Sewage Flow per person=139.2 gal.

Table 13. Chemical data raw sewage, Shawnee County S.D. 2

Date	Time	Type of Sample	5 day BOD mg/1	COD mg/1	Organic Nitrogen mg/1	NH_3 (N) mg/1	NO_3 (N) mg/1	NO_2 (N) mg/1	PO_4 mg/1	K mg/1	pH
5-22	11:30 am - 3pm	Composite	230	440	---	1.9	.30	.25	9.5	14.0	7.4
6-1	10:30 am - 3pm	Composite	57	160	---	7.5	.95	.11	.28	8.0	7.5
6-8	9:30 am -3:30pm	Composite	72	220	---	11.0	.15	.01	42.0	9.8	7.4
6-8	11:00 pm	Grab	160	320	---	14.0	.15	.01	23.0	12.0	7.5
6-9	11:00 pm -9:30pm	Composite	58	120	---	12.0	.20	.012	11.0	7.6	7.5
6-9	9:30 am - 3pm	Composite	110	290	---	12.0	.20	.018	2.4	10.0	7.3
6-9	3:00 pm -11pm	Composite	105	205	---	1.2	.55	.088	4.0	10.0	7.2
6-10	10:00 am	Grab	90	185	---	21.0	.30	.010	.59	10.0	7.5
6-10	3:00 pm	Grab	110	200	---	13.0	.33	.232	1.3	10.0	7.2
6-10	10:30 pm	Grab	100	220	---	15.0	.25	.026	4.1	11.0	7.3
6-11	10:30 pm - 9am	Composite	60	130	8.6	15.0	.30	.006	11.0	7.8	7.5
6-11	3:00 pm	Grab	128	270	---	12.0	.30	.004	33.0	10.0	7.1
AVERAGE			106.6	208.2		11.3	.33	.064	9.9	10.0	7.4

Table 14. Chemical Data Primary cells, Shawnee county S.D. 2

Date	Time	Type of Sample	5 day BOD mg/1	Filtered 5-day BOD mg/1	COD mg/1	NH_3 (N) mg/1	NO_3 (N) mg/1	NO_2 (N) mg/1	PO_4 mg/1	K mg/1	pH	D.O.
5-22	3:00 pm	Grab	50	---	150	6.2	.25	.04	.83	12.0	8.4	11.6
6-1	3:30 pm	Grab	23	13.0	110	7.2	.20	.04	.66	10.0	7.5	0.0
6-8	3:30 pm	Grab	30	10.0	140	2.4	.33	.08	6.4	7.8	9.2	23.3
6-8	11:00 pm	Grab	23	---	90	2.6	.35	.078	8.7	8.0	8.8	14.9
6-9	9:30 am	Grab	28	12.0	110	3.4	.33	.06	15.0	8.2	8.4	7.6
6-9	3:00 pm	Grab	29	---	100	4.3	.40	.068	.14	9.4	8.8	13.2
6-9	11:00 pm	Grab	18	---	70	4.5	.33	.068	.22	9.0	8.4	8.4
6-10	10:00 am	Grab	21	---	85	4.7	.33	.056	.22	8.4	8.2	5.8
6-10	3:00 pm	Grab	30	---	120	3.4	.38	.056	.38	7.8	8.7	12.3
6-10	10:30 pm	Grab	17	---	70	5.1	.33	.06	.30	8.6	8.3	7.1
6-11	9:00 am	Grab*	22	8.5	80	5.3	.35	.042	12.0	8.6	8.0	0.7
6-11	3:00 pm	Grab	28	---	100	4.9	.33	.042	13.0	8.4	8.4	10.9
		AVERAGE	26.5	10.8	102	4.5	.33	.058	4.82	8.9	8.4	9.2

*Organic nitrogen = 7.8 mg/1

Table 15. Chemical Data Effluent final cell, Shawnee County S.D. 2

Date	Time	Type of Sample	5 day BOD mg/l	Filtered 5 day BOD mg/l	COD mg/l	NH_3 (N) mg/l	NO_3 (N) mg/l	NO_2 (N) mg/l	PO_4 mg/l	K mg/l	pH	D.O.
5-22	3:00 pm	Grab	11.0	---	80	5.1	.30	.16	.60	13.0	8.0	2.8
6-1	3:30 pm	Grab	18.0	9.0	60	4.7	.10	.07	.50	11.0	7.8	0.7
6-8	3:30 pm	Grab	12.0	5.5	40	1.2	.60	.55	7.0	8.6	8.8	8.6
6-8	11:00 pm	Grab	11.0	---	40	1.5	.45	.52	6.9	9.0	8.6	5.3
6-9	9:30 am	Grab	16.0	9.5	60	1.8	.35	.34	11.0	9.4	8.4	4.6
6-9	3:00 pm	Grab	18.0	---	50	1.3	.55	.38	0.13	9.2	8.7	6.4
6-9	11:00 pm	Grab	12.0	---	45	1.1	.38	.42	0.14	8.6	8.3	4.6
6-10	10:00 am	Grab	12.0	---	55	1.2	.38	.30	0.14	8.4	8.2	4.7
6-10	3:00 pm	Grab	15.0	---	40	0.63	.55	.32	0.18	8.4	8.6	7.0
6-10	10:30 pm	Grab	11.0	---	40	0.93	.45	.34	0.24	8.6	8.3	5.8
6-11	9:00 am	Grab*	14.0	6.0	50	1.2	.35	.21	9.5	8.6	8.1	1.8
6-11	3:00 pm	Grab	13.0	5.0	50	0.63	.33	.23	9.3	8.4	8.3	6.0
		AVERAGE	13.58	7.0	50.8	1.76	.39	.32	3.8	9.2	8.3	4.9

*Organic nitrogen = 6.0 mg/l

Table 16. Bacterial data, Shawnee County S.D. 2

Influent Raw Sewage

Date	Time	Sample Number	TC/100 ml.	FC/100 ml.	FS/100 ml.
5-22	11:30am - 3:00pm	1	62 X 10^{6}	16.0 X 10^{6}	16.0 X 10^{4}
6-1	10:30am - 3:00pm	2	36 X 10^{6}	7.8 X 10^{6}	31.0 X 10^{4}
6-8	9:30am - 3:30pm	3	190 X 10^{6}	23.0 X 10^{6}	11.0 X 10^{4}
6-8	11:00 pm	4	190 X 10^{6}	11.0 X 10^{6}	41.0 X 10^{4}
6-9	11:00pm - 9:30am	5	28 X 10^{6}	6.8 X 10^{6}	53.0 X 10^{4}
6-9	9:30am - 3:00pm	6	240 X 10^{6}	11.0 X 10^{6}	20.0 X 10^{4}
6-9	3:00pm -11:00pm	7	23 X 10^{6}	17.0 X 10^{6}	11.0 X 10^{4}
6-10	10:00am	8	210 X 10^{6}	18.0 X 10^{6}	120.0 X 10^{4}
6-10	3:00pm	9	69 X 10^{6}	15.0 X 10^{6}	21.0 X 10^{4}
6-10	10:30pm	10	100 X 10^{6}	4.8 X 10^{6}	16.0 X 10^{4}
6-11	10:30pm - 9:00am	11	28 X 10^{6}	2.0 X 10^{6}	19.0 X 10^{4}
6-11	3:00pm	12	190 X 10^{6}	6.5 X 10^{6}	40.0 X 10^{4}
AVERAGES			113 X 10^{6}	11.5 X 10^{6}	37.8 X 10^{4}

Primary Cells

Sample Number	TC/100 ml.	FC/100 ml.	FS/100 ml.
1	91 X 10^{3}	22.0 X 10^{3}	410
2	930 X 10^{3}	76.0 X 10^{3}	9500
3	98 X 10^{3}	31.0 X 10^{3}	4000
4	130 X 10^{3}	83.0 X 10^{3}	5900
5	30 X 10^{3}	4.6 X 10^{3}	410
6	150 X 10^{3}	97.0 X 10^{3}	1700
7	100 X 10^{3}	26.0 X 10^{3}	1300
8	150 X 10^{3}	51.0 X 10^{3}	800
9	58 X 10^{3}	29.0 X 10^{3}	900
10	26 X 10^{3}	1.6 X 10^{3}	100
11	83 X 10^{3}	13.0 X 10^{3}	1900
12	210 X 10^{3}	36.0 X 10^{3}	3500
AVERAGES	171 X 10^{3}	35.5 X 10^{3}	253

Table 17. Bacterial data effluent final cell, Shawnee County S.D. 2

Date	Time	Sample Number	TC/100 ml.	FC/100 ml.	FS/100 ml.
5-22	3:00 pm	1	$.31 \times 10^3$	$.13 \times 10^3$	40
6-1	3:30 pm	2	39.0×10^3	3.8×10^3	1000
6-8	3:30 pm	3	3.2×10^3	$.35 \times 10^3$	110
6-8	11:00 pm	4	8.0×10^3	7.8×10^3	390
6-9	9:30 am	5	11.0×10^3	11.0×10^3	2700
6-9	3:00 pm	6	11.0×10^3	2.6×10^3	300
6-9	11:00 pm	7	4.3×10^3	$.94 \times 10^3$	80
6-10	10:00 am	8	5.6×10^3	2.0×10^3	200
6-10	3:00 pm	9	3.8×10^3	$.06 \times 10^3$	110
6-10	10:30 pm	10	2.2×10^3	$.70 \times 10^3$	50
6-11	9:00 am	11	2.4×10^3	$.40 \times 10^3$	110
6-11	3:00 pm	12	2.9×10^3	$.32 \times 10^3$	60
	AVERAGE		7.8×10^3	2.5×10^3	429

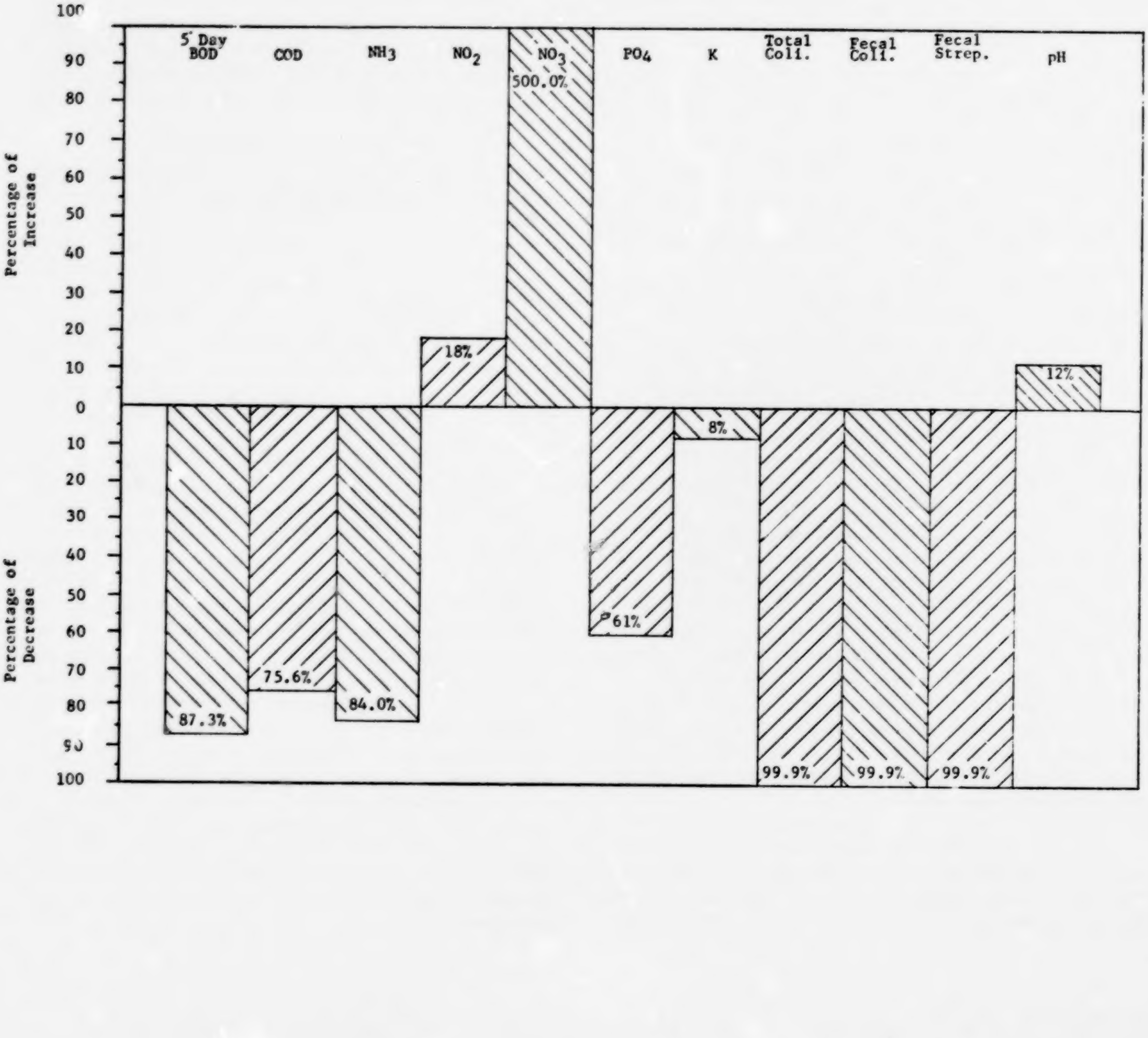
Table 18. Data summation, Shawnee county S.D. 2

	BOD$_5$ mg/1	COD mg/1	NH$_3$ as N mg/1	NO$_2$ (N) mg/1	NO$_3$ (N) mg/1	PO$_4$ mg/1	K mg/1	Total Coliform Per 100 ml.	Fecal Coliform Per 100 ml.	Fecal Strep. Per 100 ml.	pH	D.O. mg/1
Averages for Influent	106.6	208.2	11.3	.33	.064	9.9	10.0	113×10^6	11.5×10^6	37.8×10^4	7.4	--
Averages for Primary Pond	26.5	102.0	4.5	.33	.058	4.82	8.9	171×10^3	35.5×10^3	253	8.4	9.2
Averages for Effluent	13.58	50.8	1.76	.39	.32	3.8	9.2	7809	2508	429	8.3	4.9

	BOD$_5$	COD	NH$_3$ as N	NO$_2$ (N)	NO$_3$ (N)	PO$_4$	K	Total Coliform	Fecal Coliform	Fecal Strep.	pH	D.O.
Influent to Primary	75.1% R*	51.0% R	60.1% R	--	9.3% R	51% R	10% R	98.9% R	99.6% R	99.8% R	13% I*	--
Influent to Effluent	87.3% R	75.6% R	84.0% R	18% I	500.0% I	61% R	8% R	99.9% R	99.9% R	99.9% R	12% I	--

*R - Represents percentage reduction
*I - Represents percentage increase

Percentage of increase or decrease raw influent/final effluent.
Shawnee County S.D. 2



Discussion

B.O.D.–The BOD_5 removal averaged about 85%. Overbrook and Shawnee Co. S.D. No 2 gave reductions of 87.3% with the Eskridge lagoon giving 83.8% removal. The Eskridge lagoons had a much higher algae concentration due to the non-discharging nature of the final cell.

Four of the samples from the Shawnee Co. S.D. No 2 were examined for BOD_5 after filtration to remove algae. BOD_5 after filtration averaged about 10.0 mg/l from the primary cell and about 7 mg/l on the effluent from the final cell. When the samples were filtered about a 50% reduction was achieved.

C.O.D.–The results of COD analysis are much the same as BOD analysis. Reductions ranged from 60.3% removal to about 75.6% removal. COD determinations were not undertaken on the filtered samples from Shawnee Co. S.D. No. 2. COD removal was lowest on the Eskridge lagoon where the largest concentrations of algae were encountered.

Ammonia – Ammonia removal was highest (99%) on the Eskridge ponds with the longer detention time. Shawnee Co. S.D. No. 2 was next with 84.0% and Overbrook showed 79.5% removal.

Nitrate and nitrite– These two forms of nitrogen did not appear in significant quantities compared to ammonia-nitrogen in any of the samples. These analyses were run mainly as a precautionary measure as their concentrations were not expected to be of significance.

Phosphorus– Phosphorus underwent the greatest removal in the Eskridge ponds which had the highest pH (9.7). The phosphorus concentrations seemed to run somewhat lower in the raw sewage than was expected prior to the study with the Eskridge lagoon influent showing the highest average concentration of 26.7 mg/l expressed as PO_4. Shawnee County S.D. No. 2 influent had an average of 9.9 mg/l and Overbrook had an average of .677 mg/l. Phosphorus reductions ranged from almost 0.0% to 96.2%.

Coliforms– Total coliform, fecal coliform, and fecal streptococcus in all cases gave reductions greater than 99% The largest reduction occurred in the Eskridge ponds presumably because of extended detention time. The two lagoons, Overbrook and Shawnee Co. S.D. No. 2, which were loaded the heaviest showed the lesser reductions but in all cases the effluent quality was acceptable for most stream standards and the water in the secondary cell of Eskridge would meet the most stringent standards.

pH–In all three lagoons, the pH underwent a sharp rise due to the uptake of CO_2 by the algae. The pH increased in the secondary cell over the primary cell due to the larger algae concentrations.

Dissolved oxygen–The concentrations of dissolved oxygen underwent large variations. The dissolved oxygen showed the normal decrease at night with an increase in the daytime to supersaturation. Only in one case was an absence of dissolved oxygen observed. This took place in the Shawnee Co. S.D. No. 2 ponds immediately after a heavy rain which lasted intermittently during the preceding two days. This condition evidently did not last long for the pond did not produce any unusual odors but septic odors have been reported in the past.

Summary

The waste stabilization ponds serving three small communities were studied. Two of the waste stabilization ponds were overloaded according to the Kansas standards of a maximum of 34 pounds of 5-day BOD/surface acre and a 90 day detention time.

With proper maintenance and loading conditions most of the lagoons in Kansas can be expected to produce approximately an 85% reduction in BOD_5, 65% reduction in COD, 90% reduction of ammonia, an 85% to 95% reduction in phosphorus, an apparent 20% reduction of potassium and a 99% reduction or better of total coliform, fecal coliform, and fecal streptococcus. With relatively long detention times, it appears that an effluent consistently containing as little as 100 fecal coliform/100 ml. can be obtained, thus satisfying the most stringent stream standards.

It is recognized that the relationship shown between the influent raw sewage and the final effluent of the lagoon cannot be directly compared due to the detention times involved. But it can be justifiably assumed that the figures given are representative of the treatment being obtained. This is rational due to the fact that the quality of the influent sewage will probably not change much from the observed characteristics of the almost totally domestic sewage and that the detention times involved will tend to produce a stable effluent. The findings are substantiated by other data which has been published indicating similar reductions of BOD, COD, nitrogen, phosphorus, and potassium.

The natural conditions in Kansas streams due to grassland runoff, forested area runoff, and agricultural land runoff show that water quality conditions in Kansas streams are such that effluent discharge from series operated lagoons will enhance overall quality characterisites. It should also be indicated that effluent characteristics from these lagoons were determined from unfiltered samples and represents the total loads imposed upon a receiving stream. In many areas of the state it is not uncommon practice at the present time to utilize lagoon effluents for the sole source of water supply for livestock watering purposes and general crop irrigation. Under these described conditions it would not be impertinent to suggest that waste stabilization ponds are highly superior to conventional waste treatment facilities as it relates to overall water quality characteristics and with possible only one or two exceptions it would be foolish to consider advanced waste treatment with the removal of nitrogen and phosphorus from conventional treatment systems within metropolitan areas.